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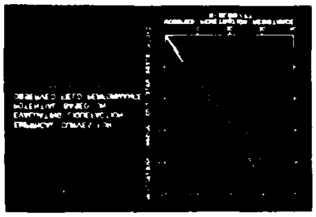

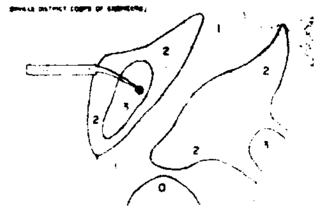

GEOTECHNICAL LABORATORY

(2)

TECHNICAL REPORT GL-86-7

SEISMIC STABILITY EVALUATION OF ALBEN BARKLEY LOCK AND DAM PROJECT

Volume 3

FIELD AND LABORATORY INVESTIGATIONS

by

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Geotechnical Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39181-0631

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The Alben Barkley Lock and Dam Project, located on the Cumberland River, approximately 25 miles upstream of Paducah, Kentucky, has been the subject of extensive field and laboratory investigations designed to provide stratigraphic and strength information essential to the completion of a seismic stability evaluation of the project, deemed necessary since the project lies near the boundary between Seismic Zones 2 and 3 as defined in Engineer Regulation 1110-2-1806. This report is one of a series of reports pertaining to this evaluation. It is primarily a geotechnical data report. Analysis of the data is presented in later reports in this series.

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PREFACE

The US Army Engineer Waterways Experiment Station (WES) was authorized to conduct this study by the US Army Engineer District, Nashville (ORN), by Intra-Army Order for Reimbursable Services Nos. 77-31 and 77-112. This report is Volume 3 of a 5-volume set which documents the seismic stability evaluation of Alben Barkley Dam and Lake Project. The 5 volumes are as follows:

Volume 1: Executive Summary

Volume 2: Geological and Seismological Evaluation

Volume 3: Field and Laboratory Investigations

Volume 4: Liquefaction Susceptibility Evaluation and Post-Earthquake Strength Determination

Volume 5: Stability Evaluation of Geotechnical Structures

The work in this volume is a joint endeavor between ORN and WES. Mr. Paul F. Bluhm, of the Geotechnical Branch at ORN, coordinated the contributions from ORN. Mr. Richard S. Olsen and Dr. M. E. Hynes, of the Earthquake Engineering and Geophysics Division (EEGD), Geotechnical Laboratory (GL), WES, coordinated the work by WES. The preliminary stages of this project were conducted by Dr. William F. Marcuson III, who was Principal Investigator from 1976 to 1979. From 1979 to project completion, Dr. M. E. Hynes was Principal Investigator. Mr. Bluhm was assisted in this study by Mr. Joseph J. Melnyk, geologist (ORN). The geophysical field studies were conducted by Mr. Robert F. Ballard, Jr., and Mr. Donald E. Yule, GL, WES. Mr. Yule prepared the results of these studies for this report. Overall direction at WES was provided by Dr. A. G. Franklin, Chief, EEGD, and Dr. Marcuson, Chief, GL.

Overall direction at ORN was provided by Mr. James E. Paris, Chief, Soils and Embankment Design Section; Mr. Marvin D. Simmons, Chief, Geology Section; and Mr. Frank B. Couch, Jr., Chief, Geotechnical Branch. Mr. E. C. Moore was Chief, Engineering Division. COL Edward A. Starbird, EN, was District Commander.

Technical Advisors to the project were Professors H. B. Seed (University of California, Berkeley), Alberto Nieto (University of Illinois, Champaign-Urbana), and L. Timothy Long (Georgia Institute of Technology), and Dr. Gonzalo Castro (Geotechnical Engineers, Inc.). Comments from Drs. Seed and Castro regarding the liquefaction evaluation and stability analyses are appended to Volume 4 of this series.

COL Dwayne G. Lee, EN, was Commander and Director of WES. Dr. Robert W. Whalin was Technical Director.

CONTENTS

* Limited number of copies of Appendixes A-J were produced under separate covers.

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
feet	0.3048	metres
inches	2.54	centimetres
miles (US statute)	1.609347	kilometres
pounds	4.448222	newtons
pounds per square inch	6.894757	kilopascals
square miles	2.589998	square kilometres
tons per square foot	95.76052	kilopascals

SEISMIC STABILITY EVALUATION OF ALBEN BARKLEY LOCK AND DAM PROJECT

Field and Laboratory Investigations

PART I: INTRODUCTION

1. The Alben Barkley Lock and Dam Project, located on the Cumberland River, approximately 25 miles upstream of Paducah, Kentucky, has been the subject of extensive field and laboratory investigations designed to provide stratigraphic and strength information essential to the completion of a seismic stability evaluation of the project, deemed necessary since the project lies near the boundary between Seismic Zones 2 and 3, as defined in Engineering Regulation 1110-2-1806. This report is one of a series of reports pertaining to this evaluation. It is primarily a geotechnical data report. Analysis of the data is presented in later reports in this series.

2. This report documents the results of a brief examination of the geological history of the area, pertinent information obtained from design and construction records, observations of pool elevations since completion of the dam in 1964, and the main findings of field and laboratory geotechnical and geophysical investigations performed during the years 1977 to 1985.

3. The project consists of a concrete gravity dam, powerhouse and lock system 109 ft tall at maximum section, founded on limestone and flanked by homogeneous compacted rolled-fill earth dams. The embankment dams are about 8,700 ft in total length and 55 ft tall at maximum section, and are founded on an alluvial deposit with a maximum thickness of approximately 120 ft and underlain by limestone. The alluvium, a complex layering of clays, silts, sands and gravels, is the focus of concern in the seismic safety assessment due to the possibility of liquefaction of these sediments during an earthquake. The objectives of the field and laboratory investigations are to provide sufficient information to estimate the response of the dam and foundation to earthquake ground motions, to measure the resistance to liquefaction of the many types of soils present in the alluvium, and to provide sufficient stratigraphic detail so that the areal extent of possible problem zones can be estimated, and informed stability decisions can be made.

4. To accomplish these objectives, a wide variety of field investigation techniques was employed, namely, geophysical tests, Standard Penetration Tests (SPT), Cone Penetration Tests (CPT), undisturbed sampling, Wissa probe soundings, and excavation of streambank sediments for geological mapping. In the laboratory, tests included sieve and hydrometer analyses, Atterberg limits, specific gravity, laboratory vane shear, pocket penetrometer, and triaxial tests. The most effective technique for developing an understanding of the site stratigraphy was the CPT, supported by the excavation and SPT. The CPT was also a key to determining liquefaction resistance and post-earthquake strength (described in Volume 4 of this series), supported by the laboratory triaxial tests. Consequently, the field CPT procedures are described in detail in this report. Tabulated and plotted data are contained in the appendixes.

PART II: PROJECT DESCRIPTION

General

5. The Barkley Project is located on the Cumberland River, 30.6 miles above its confluence with the Ohio River. It is situated in Livingston and Lyon Counties, Kentucky, near Grand Rivers, Kentucky, 25 miles east of Paducah, Kentucky, and 160 river miles below Nashville, Tennessee (see Figure 1). The reservoir extends 118 miles upstream to Cheatham Lock and Dam Project, located near Ashland City, Tennessee. The multi-purpose Barkley Project is a key unit in the comprehensive plan of development of the Cumberland River. It provides flood control, hydroelectric power, navigation, and recreation. The reservoir is contained by an concrete gravity section flanked by earth embankment dams. The concrete section includes a gated spillway, a lock, and a power house. The dam supports a railroad track system which traverses most of the dam crest. A canal, large enough for barge traffic, connects Barkley and Kentucky Lakes about 2.5 miles upstream from the dam. At the maximum flood control pool, Elevation 375 ft, the reservoir stores 2,082,000 acre-ft, with 13 ft of freeboard (minimum crest Elevation 388 ft). For normal operation, the pool elevation varies from 354 to 359 ft, and stored volume varies from 610,000 to 869,000 acre-ft, respectively. These main elements of the project are described in the following paragraphs.

Right Embankment Dam

6. The right embankment dam is a homogeneous, rolled-earth, compacted impervious fill, with a downstream drainage blanket. Figures 2 and 3 show plans and sections of the dam. The embankment is founded on a deep deposit of alluvium, which has a maximum thickness of about 120 ft and is underlain by limestone. The length of the right embankment is about 7,116 ft. The upstream slopes are 1 vertical to 2.5 horizontal from the upstream toe of the dam to Elevation 380 ft, and 1 vertical to 2 horizontal from Elevation 380 ft to the top of the dam. The downstream slopes are 1 vertical to 2 horizontal from the dam crest to Elevation 375 ft, and 1 vertical to 4.5 horizontal from Elevation 375 ft to the downstream toe of the slope. A 2-ft thick drainage

blanket extends from 20 ft downstream of the dam's centerline to a rock toe drainage ditch. Figure 3 shows the detailed section.

7. The width of the crest is 22 ft from the connection with the right end of the powerhouse, Station 33+52L, to Station 44+02L, where a transition zone 129 ft long begins as the crest width is increased to 37 ft, to accommodate the transition from a single- to a double-track railroad system. The 37-ft crest width continues to the right abutment of the embankment, Station 104+68L. The right embankment crest elevation has a maximum of 394.5 ft at the powerhouse, decreases with a 0.5 percent slope to Elevation 388.0 ft at Station 51+50L, and remains at Elevation 388.0 ft from Station 51+50L to the right abutment. Typically, the embankment height is about 55 ft near the powerhouse and 40 ft elsewhere along its length. Figures 2 and 3 show the detailed plans.

8. A switchyard and access roads are located downstream of the centerline from the powerhouse to Station 44+00L. The switchyard is on a large, fairly level berm, with a surface elevation of 366 ft, that extends about 370 ft downstream of the dam's centerline. An inclined drain was added to control seepage in this area. The drain is 9 ft wide (horizontal measurement) and starts at the centerline at Elevation 370 ft. It has a slope of 1 vertical to 1.5 horizontal and connects to the horizontal drainage blanket. Figure 2 shows details of this section. A sheetpile cutoff was driven through the natural alluvium to rock and a grout curtain was constructed from Station 33+81L to Station 38+52L. Retaining walls were built upstream and downstream of the powerhouse, parallel to the direction of flow, to protect the embankment dam and its alluvial foundation as these materials slope down to the spillway and tailrace foundation excavation elevations, approximately Elevation 255 ft. Figures 4 and 5 show sections of the sheetpile cutoff, grout curtain, and retaining walls.

Left Embankment Dam

9. The plan and sections of the left embankment are shown in Figures 6 and 7. This embankment dam is about 1,600 ft long and is composed of a compacted impervious rolled fill with a section of select pervious fill on the upstream face (see Figure 7 for typical section). The depth to rock in this area is relatively shallow, typically 40 ft or less, so the foundation soils

were excavated to rock, from Station -0+65 at the left abutment to Station 14+22 at the connection of the left embankment with the landward lock wall. The core trench is 10 ft wide at rock level and the slopes in the natural alluvium on either side of the trench are 1 vertical to 1.5 horizontal. The exposed limestone bedrock received careful dental treatment and was grouted. The embankment is typically 40 ft in height with a crest elevation of 388.0 ft, and a crest width of 30 ft. The upstream slopes are 1 vertical to 3 horizontal, and the downstream slopes are 1 vertical to 2.5 horizontal. Random fill, described in construction documents to be saturated, fine-grained soils, were placed on the upstream slope to Elevation 366 ft, and on the downstream slope to Elevation 381 ft. The construction records imply that no provisions for drainage were made in the downstream area, except along the lock wall (see COMPLETION REPORT, LEFT BANK COFFERDAM, Volume 2, dated May 1959, Pages 32-43).

Concrete Structures

10. The concrete gravity section, which is founded entirely on limestone bedrock, includes the spillway, the powerhouse, and the lock. The overflow section is 804 ft long and has 12 gated spillways. With gates in the fully closed position, the elevation of the top of the gates is 375 ft, and the elevation of the bottom of the gates is 325 ft (which is the spillway crest elevation with gates open). The maximum concrete section height above the streambed is 157 ft. The powerhouse section is 430 ft long and the lock section is 221 ft long. The clear dimensions of the lock chamber are 110 ft by 800 ft, with a normal lift height of 57 ft. The elevation of the top of the lock's walls is 382 ft. The Barkley Lock was placed in operation in 1964. The powerhouse, with 4 generating units of 32,500-kw capacity each, was placed in operation in 1966.

Canal

11. A canal, large enough for barge traffic, connects Kentucky and Barkley Lakes about 2.5 miles upstream from the dam. The canal is 1.75 miles long, 400 ft wide at the bottom (Elevation 335 ft), and 11 ft deep at minimum pool (Elevation 346 ft). No gates were constructed to regulate flow through

the canal. The canal is crossed by Route 453 which runs north-south along the narrow strip of land between the two lakes. Kentucky Lake has much more storage capacity than Barkley Lake, so the canal is a critical element in the assessment of downstream hazard potential.

PART III: GEOLOGY OF THE PROJECT AREA

Regional Geology

General

12. The Barkley Project is located in the extreme northern part of the Mississippi Embayment, which extends over an area of about 100,000 square miles in the Gulf Coastal Plain, as shown in Figure 8. The Mississippi Embayment fans out southward from southern Illinois to about the 32nd parallel and includes parts of Alabama, Arkansas, Illinois, Kentucky, Louisiana, Mississippi, Missouri, Tennessee, and Texas. The geology, development, and geologic history of the Mississippi Embayment along with the site geology are briefly summarized in the following paragraphs to assist in the overall understanding of the Barkley Dam site and in the identification of materials of particular concern in the seismic safety assessment of the project.

Geology of the Mississippi Embayment

13. Mississippi Embayment structural features modify the embayment somewhat, but it is essentially a downwarped trough or syncline of Paleozoic rocks in which sediments ranging in age from Jurassic to Recent have been deposited. See geologic time scale in Figure 9. The axis of the trough plunges to the south and roughly follows the present course of the Mississippi River. The greatest thickness of post-Paleozoic sediments or rocks filling the trough is approximately 18,000 ft and occurs in the extreme southern part of the embayment, in the area of greatest subsidence and downwarping. The sediments generally are sands, silts, clays, gravels, and chalks.

Development of the embayment

14. The downwarping and subsidence that formed the Mississippi Embayment was probably caused by subcrustal or tectonic movement of the Paleozoic rocks, by sedimentary loading of the Paleozoic rocks, and by compaction of the sediments filling the embayment. The initial subcrustal movement may have been associated with the Appalachian revolution at the end of the Paleozoic Era. Uplifted structures, such as the Ouachita Mountain system, the Ozark uplift, and the southeastern extremity of the Appalachian Mountain system that occupied the area at the beginning of the Mesozoic Era, were sources of large amounts of various sediments that were deposited in the rudimentary embayment. The deposition resulted in sedimentary loading of the underlying Paleozoic

rocks which caused or aided further downwarping of the trough. It also resulted in subsidence from the compaction of the accumulating sediments. The deposition occurred concurrently with subsidence and inundation of the trough of the embayment and the Gulf Coast geosyncline.

Geologic history

15. The geologic history is discussed in terms of geologic time units. A geologic time chart with a geologic column is given in Figure 9.

Paleozoic Era

16. During the Paleozoic Era, a changing sea covered most of the interior of North America. This changing sea deposited sediments during times of submergence. These sediments were then partially or completely eroded during times of exposure, all of which resulted in variably alternating shales, sandstones and limestones, ranging in age from Cambrian to Pennsylvanian. The end of the Paleozoic Era was marked by an extensive period of erosion leaving an irregular surface onto which the Mesozoic sediments were deposited.

Mesozoic Era

17. The Mississippi Embayment was essentially formed during the Mesozoic Era. Some subsidence occurred in the embayment during the Jurassic Period; however, it was greatest during the Cretaceous Period. As the land subsided, the Cretaceous sea advanced northward, depositing more and more sediments in the embayment. A major retreat of the sea occurred during the Cretaceous separating it into the early and late Epochs. It was during the Late Cretaceous Epoch that the sea extended its maximum distance to the north. Various structural features came into existence during the Late Cretaceous Epoch which essentially resulted in the embayment's present size and configuration.

Cenozoic Era

18. Cyclic advances and retreats of the sea dominated the Tertiary Period. Marine rocks of the Paleocene and Eocene Series can be found in the extreme northern part of the embayment indicating extensive inundation by the sea during these epochs. Some Oligocene and Miocene deposits can be found in the southern part of the embayment; however, most of the Mississippi Embayment has been above sea level since the end of the Eocene Epoch. Some subsidence and adjustment continued during the Quaternary Period. The Mississippi River Valley's terraces were formed and alluvial fill deposited during the Quaternary Period. The Pleistocene Epoch furnished huge amounts of sand, gravel,

clay, and loess from the melt water of glaciers that occupied the area north of the embayment. Cumberland River alluvium was deposited during the Recent Epoch of the Quaternary Period. The various modifying structural features and erosion of the ancient sea bottom has resulted in the embayment's present topography.

Site Geology

General

19. Being located near the margin of the Mississippi Embayment, the Cumberland River, in the area of the Barkley Project, has completely cut through the continental and marine sediments that once completely filled the embayment and has incised itself into the underlying trough of Paleozoic rocks. See the generalized geologic cross section in Figure 10. Remnant outcrops of the embayment sediments are found capping the hills and ridges in the area while the valley slopes are comprised of Paleozoic rocks. Alluvium is present in the valley bottoms of all major streams and rivers. The concrete structures for the Barkley Project were founded in the Mississippian Warsaw formation while the earth embankments were founded on alluvium.

Alluvium

20. Much discussion and importance, as related to foundation stability, were given to the question of whether the materials under the right embankment were alluvial deposits or lacustrine. Lacustrine deposits are generally continuous over large areas while alluvial deposits are irregular and discontinuous in both plan view and elevation. The environment for each is described below.

21. Alluvium is deposited in stream channels, floodplains, and in alluvial fans at the mouth of the stream. The subject material at Barkley Dam is a floodplain deposit. Kinetic energy of the stream or river and the boundary conditions of stream gradient, linear shape of the channel, and limiting valley walls determine the alluvial environment. The processes are predominantly physical in the alluvial environment, as opposed to chemical or biological. Energy of the stream or more precisely of the flowing water governs the size of the particles transported and the amount of sorting. The turbulent flow of streams develops a high degree of selection in the load that is being carried, but this is offset by daily or seasonal changes in velocity and turbulence.

This process tends to produce lenticular beds with different size characteristics. The alluvial deposits develop as elongated lenses oriented generally downstream in the direction of greatest flow energy. With regard to a lacustrine environment, the boundary conditions of lakes include their size, shape, and depth of water. Large lakes, which an ancient lake at Barkley site would have been if it existed, may have sufficient wave energy to develop well marked shore features. Bottom deposits of a large lake would consist of fine sand, silt, and clay that would be derived from the shore deposits and would be mixed with organic matter and any chemical precipitates that may have formed, most commonly calcium carbonate. These bottom deposits commonly show some sorting and lamination. In lakes with regular overturn, the laminations are especially uniform. With these two scenarios in mind, a bank exposure was mapped in materials that were considered to be similar to those in the foundation. See Part IX for details of this study. Based on the relative thicknesses, the lenticular nature, the discontinuity of many of the beds that were seen and mapped in the exposure, and on the undulating and uneven boundaries between the beds, it was concluded that the foundation materials in question were alluvial in nature and not lacustrine.

22. The primary geomorphic features in the Cumberland River Valley in the area of Barkley Dam consist of a river channel, a natural levee on each side of the channel, and a floodplain with an undulating surface. The undulations on the floodplain are elongated in an upstream/downstream direction. There is an absence of meander scars on the floodplain. The narrow and confining nature of the valley and the underlying influence of the limestone bedrock apparently prevented the river from developing large, looping meanders and subsequent meander cutoffs.

23. A typical profile of the alluvium can be divided into three main zones or units as shown on Figure 10. The first zone, Unit 1, extends from the ground surface to a depth of 10 to 20 ft and is generally made up of a medium stiff clay with low to moderate plasticity. This material is an overbank deposit laid down on the floodplain during times of flooding. The second zone, Unit 2, extends from the bottom of Unit 1 to a depth of 50 to 60 ft and consists of a highly stratified sequence of clays, silts, and sands as well as mixtures such as silty and clayey sands, clayey silts, and silty and sandy clays. These overbank deposits range widely in grain size, thickness, and areal extent. Unit 3 extends from the bottom of Unit 2 to a depth of 120 ft

and consists of gravels and denser sands and silty sands with some layers of clay also being present. These materials are channel deposits laid down as the river swept across the valley. The different depositional environments for each of the three units described above probably resulted from changing baselevels that occurred in the geologic past.

24. The alluvium at Barkley has not been preconsolidated from any overlying glacial ice as the advance of the glaciers essentially stopped at the present location of the Ohio River about 15 miles to the northwest. In addition, the alluvium was deposited subsequent to glaciation.

Loess

25. The loess deposits in the area of the damsite have not been mapped by the United States Geological Survey; however, they consist of predominantly wind blown silt. Thin loess deposits mantle much of the general area, but no deposits of loess, as such, have been identified under the dam.

Terrace gravels

26. Terrace gravels of Pleistocene age and possibly some Pliocene age gravels are present primarily at elevations above the alluvium. Some terrace gravels are thought to be present on the section under the right abutment as shown in Figure 10. These terrace deposits generally consist of sandy gravel and cobbles. This material is somewhat lithified, and in places, it is well cemented with iron oxides. Any bedding present in these materials is not well defined. These materials were probably laid down by the Cumberland River when it was at a higher elevation.

McNairy Formation

27. The McNairy Formation is a marine material that was deposited in the ancient Cretaceous sea that was present in the Mississippi Embayment. This formation has been completely cut through by the rivers and streams in the area of the dam and can only be found capping the nearby hills and ridges. The formation is primarily comprised of fine-grained sands with thin interbeds of silt and clay.

Tuscaloosa Formation

28. The Tuscaloosa is a gravel having cobbles and a slight matrix of clay, silt, and sand. Except for some cross bedding, bedding is uncommon. Some silica cementation is present locally at the top of the formation. This formation is found in the hills and ridges above the valley bottoms.

St. Louis Formation

29. For the purposes of the Barkley Project, the St. Louis is undifferentiated from the Salem limestone. The St. Louis is present in the lower reaches of the valley walls in the area of the damsite and consists of variably argillaceous limestones. No structural foundations of the dam were in the St. Louis.

Warsaw Formation

30. The Warsaw is the foundation rock for the concrete lock and spillway portions of the dam. This relatively pure limestone is fossiliferous and weathers readily. Many solution channels and cavities were present in this formation.

Fort Payne

31. A cherty, argillaceous limestone comprises the Fort Payne formation. The powerhouse is founded on this limestone. Some solutioned joints were found in the Fort Payne during construction.

Karstic limestone in foundation rock

32. The defects in the limestone foundation rock are the result of jointing and weathering, especially by solution. There were two systems of principal joints in the rock at 90 degrees to each other, both basically vertical, each system crossing the river at approximately 45 degrees. There also appeared to be a secondary joint system roughly parallel to and perpendicular to the river. The joints were of importance, not because they were a serious foundation defect in themselves, but because they controlled solutioning and weathering. Another factor regulating solutioning of foundation rock is that the pure limestone of the Warsaw formation is more soluble than the argillaceous, cherty Fort Payne limestone. Thus, with few exceptions, solution channels tend to pinch out near the Warsaw-Fort Payne contact. Solutioning along horizontal bedding planes, a much worse condition in terms of foundation stability, was minimal and did not pose a problem for the structures. All solutioned joints encountered in the foundations for the powerhouse adjacent to the right embankment, as well as the other concrete structures, were excavated out and backfilled with concrete (dental treatment). The only exception to this kind of treatment was for a portion of the downstream guide wall of the lock where very large and deep solution channels were encountered. This badly solutioned area was bridged over with concrete. Solutioned rock, determined from core borings, was also encountered beneath the alluvium under the right

embankment. No treatment of this rock was done during construction, apparently because of the rock's substantial depth below the surface. Problems in the right embankment area because of the solutioned rock have not developed to date.

Faulting

33. The Barkley Project is located in Seismic Zone III (Stearns, 1978) about 71 miles from the source area of the New Madrid earthquakes that took place in 1811 and 1812. Stearns concluded in his report that there were no active faults at or near Barkley Dam. Although not active, a number of faults have been identified in the general area. One fault crosses the Cumberland River less than a mile and a half upstream of the dam. Two others cross downstream, one at about four miles and the other at five. The nearest fault to the west is about a mile and a half, while to the east a number of faults are present a couple of miles away. Areas further beyond Barkley are heavily faulted. For a detailed account of the seismic hazard, see Volume 2 of this report series (Krinitzsky, 1986).

PART IV: REVIEW OF DESIGN AND CONSTRUCTION RECORDS

Design Investigations and Records

Pre-construction field investigations

34. For design of the earth embankments, the pre-construction boring program along the centerline of the dam consisted of 18 drive sample holes (churn rig), generally on 400-ft centers, and 2 undisturbed Denison holes. In the areas upstream and downstream of the powerhouse, 24 drive sample holes (churn rig) and 2 undisturbed Denison holes were also drilled. Numerous probings, auger and washbore holes were also made. See Figure 11 for locations of these explorations. No SPT tests were conducted in any of these borings.

Pre-construction laboratory testing

35. Laboratory testing on the drive samples consisted of sieve analysis, Atterberg limits, and natural moisture content. Only a few selected samples were tested and much of the soil was visually classified in the field. The above tests were also performed for the undisturbed samples along with specific gravity, dry density, shear strength, and permeability. Table 1 summarizes the test results of the saturated and dry densities and the shear strengths. Zones A, B, and C noted in Table 1 correspond approximately to Units 1, 2, and 3, respectively, of the foundation described in paragraph 23. The test results shown in Table 1 were used for the design of the dam.

Construction of Dam

Construction

36. Construction of the dam began in 1961 with the right embankment and switchyard being built in two phases. The first phase was construction of 800 ft of the embankment, switchyard, and pervious drainage blanket up to Elevation 360 ft. Material used for this phase was obtained from the powerhouse excavation. A permanent sheet pile cutoff wall was also constructed in this phase which extended from the powerhouse to Station 38+52L. The wall was driven to rock and had a top elevation of 325. The second phase of construction began in 1962 and consisted of building the remainder of the embankment and switchyard. A cutoff trench, 10 by 10 ft, was excavated for the entire length of the right bank. Materials used for the embankment were obtained

from several borrow areas upstream of the dam. A lean, silty clay was used for sections of the impervious embankment and switchyard and was compacted in 4- to 8-in. layers with 6 passes of a 10-ton sheepsfoot roller. The pervious drainage blanket was a crushed limestone aggregate with a topsize of 1-1/2 in., D_{50} of 1/2 in., and not more than 5 percent passing the No. 200 sieve. This material was compacted by the hauling and spreading equipment. The only serious problem encountered was excessive settlement in the switchyard near a cable tunnel. Problems relating to the dam and/or foundation during construction are described below:

a. Excessive settlement occurred in the switchyard near the cable tunnel. Poor compaction along the tunnel was believed to be the cause of the settlement and the material was removed and recompacted.

b. Sinkholes occurred in the overburden slope at Station 38+00L and 1+40A. This area was excavated to rock and a solution channel was located. A dewatering system was then employed so that the excavation could be backfilled. The solution channel was then backfilled with grout through the dewatering pipes.

c. A slide occurred along a cut slope near Stations 29+77L and 13+37B. A haul road had been constructed about half way up the slope, which coupled with the 1 horizontal to 1 vertical slope, was determined to be the cause of the slide. An undisturbed Denison boring was drilled at this location to correlate with the preconstruction boring BDH-10, located nearby. No correlation of individual sand layers was possible. Results of the testing of the samples from this hole is summarized in Table 2.

37. Sampling and testing. In the first phase of construction, 64 field density tests were made and 7 record samples taken, 6 of which were of the foundation material and one of the embankment. For the second phase, 409 field density tests and 12 record samples were taken. The results of the field density tests showed that the average dry density and water content was 106.5 pcf, 19.2 percent and 107.3 pcf, 17.8 percent, for Phases 1 and 2, respectively. Tests performed on the record samples included sieve analysis, Atterberg limits, natural water content, dry density, specific gravity, strength, permeability, and consolidation. Pertinent data for the two phases are summarized in Tables 3 and 4.

PART V: POOL LEVELS

38. For normal operations (not flood conditions) the reservoir level at the dam follows a guide curve (see Figure 12). Elevation 354 ft is maintained from 1 December through 31 March. The pool is then gradually raised to Elevation 359 ft during April where it is maintained to about 1 July and thereafter it is gradually lowered back to Elevation 354 ft by 1 December. Flood control storage extends up to Elevation 375 ft; however the maximum flood of record to date is about Elevation 370 ft. Data gathered since 1968 show that the reservoir level has exceeded Elevation 361 ft about 4 percent of the time or an average of about 2 weeks per year. Figure 13 shows the annual probability of exceeding Elevation 361 ft plotted against the pool elevation. The tailwater at Barkley Dam is controlled by downstream structures located on the Ohio River. Minimum tailwater elevation is 302. However, historical records show that the tailwater elevation can range between 320 and 340 in the winter and spring months and from 302 to 318 in the summer and fall months.

39. The headwater and tailwater elevations used for the seismic stability analyses of Barkley Dam were 360 and 305, respectively. The probability of the simultaneous occurrence of the maximum design earthquake and a flood which brings the headwater elevation to a significant level above the normal reservoir level (a 5-year frequency flood raises the pool level to Elevation 365) is very small. Therefore, the headwater elevation selected for the analysis was 360. For the tailwater an elevation of 305 was used. Stability analyses indicate that the critical conditions exist when the tailwater is at a minimum. Since the tailwater can be at Elevation 305 ft for half of the year, this elevation was selected to be used in the analysis.

PART VI: GEOPHYSICAL SURVEYS

General

40. The purpose of the geophysical surveys was to measure the shear-wave (S-wave) velocity, V_s , and the compressional-wave (P-wave) velocity, V_p , of the embankment and foundation soils from the ground surface to bedrock. Although the V_s profiles are only low resolution indicators of stratigraphy (layers on the order of a few feet in thickness can be resolved) they are the dominant input parameter in dynamic response calculations for a given earthquake and dam and foundation geometry. Consequently, V_s measurements were made at five areas to detect variations in V_s profiles along the axis of and perpendicular to the dam. The V_p profiles are used primarily to distinguish between saturated and partially saturated soil zones.

41. Crosshole, downhole, P-wave surface refraction, S-wave surface refraction, and Rayleigh wave tests were performed. Specially instrumented cone penetrometer test (CPT) equipment was used for the downhole tests at two of the study areas. By far the most accurate measurements are made with crosshole tests, so these results were given the most weight in the development of velocity profiles for the dynamic response analyses. At two key locations, namely the dam centerline and the center of the switchyard, it was not possible to conduct crosshole tests for reasons such as accessibility, traffic logistics, technical problems such as interference from switchyard equipment, and cost. The V_s profiles were estimated in these cases from the closest reliable measurements adjusted for confining stress differences.

42. The five locations examined with surface and subsurface geophysical methods to measure V_s and V_p profiles are: (a) Location 1, Station 64+00L, Offset 2+40B, a three-hole crosshole set with 3 refraction lines and 1 Rayleigh wave line at the downstream toe of the dam, and 1 refraction line along the crest of the dam, (b) Location 2, Station 36+00L, Offset 0+39B, a two-hole crosshole set near the edge of the service road, on the downstream slope of the embankment, (c) Location 3, Station 34+45L, Offset 4+95B, a two-hole crosshole set at the downstream toe of the switchyard, (d) Location 4, Station 38+70L, Offset 2+07B, downhole tests at CPT 12, and (e) Location 5, Station 34+56L, Offset 4+98B, downhole tests at CPT 26. Figure 14 shows the locations of these test areas on a plan of the right embankment, Figure 15

shows a detailed plan of the test layout at Location 1, Figure 16 shows a detailed plan of the test performed in the switchyard area, and Table 5 summarizes descriptive information about the types of tests performed. The tests at Location 1 were conducted in 1977, the tests at Locations 2 and 3 were conducted in 1984, and the CPT work was done in 1985.

Preparation of Crosshole Test Areas

43. At the three crosshole test locations, borings were drilled 8 in. in diameter and cased with 4-in. ID PVC pipe. The annular space between the casing and the walls of the borings was grouted with a special grout mixture designed to have the consistency of soil after setting up. At test Location 1, Station 64+00L, a set of three borings in a triangular array was drilled. A two-boring set, with borings spaced approximately 10 ft apart, was drilled at test Locations 2 and 3 (Station 36+00L and Station 36+45L, respectively). A borehole deviation survey of each boring was conducted to determine precise vertical alignment since accurate reduction of data from the crosshole test requires knowledge of the drift of each borehole. With this information, the straight-line distance between boreholes at each test depth can be accurately determined. The top-of-hole elevations were surveyed to assist correlation with other borings.

Test Procedures

44. Detailed descriptions of geophysical field procedures are given in EM 1110-1-1802 (1979). Summary details at individual test locations are mentioned below.

Crosshole S-wave tests

45. At Location 1, the S-wave crosshole tests were performed with a surface-mounted vibrator which transmitted vertically polarized waves by means of a pipe connected to the vibrator at the surface, extended inside the PVC casing, and coupled with the casing at the selected testing depth (Ballard, 1976). Next, a triaxial geophone array was lowered into the other borehole to the same elevation. When the vibrator and receiver were in position, the operator swept the oscillator through a range of frequencies (50 to 500 Hz) and selected one that propagated well (one with a high amplitude and

nondisturbed waveform) through the transmitting medium. The time required for the transmitted signal to reach the receiver geophone was recorded with a seismograph without enhancement capabilities. Measurements were made at 10-ft intervals.

46. The S-wave crosshole test procedures at Locations 2 and 3 were similar to those used at Location 1 except the the S-wave source was a downhole vibrator which was lowered into the hole at selected test depths and firmly attached to the sidewalls of the borehole by means of an inflatable rubber bladder. The downhole vibrator transmitted vertically polarized shear waves. The time required for the transmitted signal to reach the receiver geophone was recorded with a seismograph with enhancement capabilities. Tests conducted at Location 2 were at 5-ft-depth intervals, and those conducted at Location 3 were at 2.5-ft-depth intervals.

47. The field data was processed with the computer program CROSSHOLE (Butler et al., 1978) to calculate true in situ V_s and V_p values, to identify relatively uniform velocity zones, and to determine depths to interfaces between zones of different velocities.

Crosshole P-wave tests

48. The crosshole P-wave tests were conducted in a manner similar to the S-wave tests except that exploding bridgewire detonators (EBW's) were used as the P-wave source. Crosshole P-wave measurements were conducted at 10-ft intervals at Location 1. Tests conducted at Location 2 were at 5-ft-depth intervals, and those conducted at Location 3 were at 5-ft-depth intervals until a depth of 35 ft at which point they were run every 2.5 ft to the bottom of the holes. The CROSSHOLE program was used to process these results.

Downhole tests

49. Downhole P-wave and S-wave tests were performed at Locations 1, 4, and 5. The polarized shear wave source consisted of a wooden plank secured near the top of the borehole and struck on either end with a sledge hammer to generate horizontally polarized waves. The plank was offset 1 ft from the edge of the borehole at Location 1, and 20 ft from the CPT rods at Locations 4 and 5, to minimize direct transmission of waves down the rods. Measurements were made at 10-ft intervals at Location 1, and at 5-ft intervals at Locations 4 and 5. The P-wave source was a sledge hammer impact to a steel plate on the ground surface. A more extensive description of the CPT

instrumentation and procedures for downhole seismic testing is given in the ERTEC (1985) report in Appendix F.

Surface refraction
and Rayleigh wave tests

50. As shown in Figure 15, 4 surface refraction lines were run at Location 1. Lines RS-1 and RS-2 were run on either side of the crosshole set, parallel to the axis of the dam. Line RS-2 was run perpendicular to the axis of the dam, just downstream of the crosshole set. Line RS-4 was run along the crest of the dam near Station 64+00. The downstream lines were 625 ft long and the crest line was 165 ft long. Forward and reverse traverses were made on each line. The P-wave ground response was monitored with 24 vertically oriented geophones spaced at 15-ft intervals along a straight line. Response was recorded on a portable battery operated 24-channel seismograph and oscillograph. The P-wave seismic energy source for the lines at the toe of the dam was provided by detonation of explosives (1 to 2 lb) in shotholes 10 ft deep. A sledge hammer impact on a steel plate was used as the energy source on the embankment.

51. In addition to the P-wave procedures described above, refracted S-wave tests were conducted along RS-3 and RS-4 by replacing the vertical geophones with horizontal units oriented perpendicular to the test line. A wooden plank, secured to the ground, and struck on either end with a sledge hammer was the S-wave energy source.

52. As an additional check on near surface V_s measurements of the foundation materials, 22-ft-long Rayleigh wave line was run at the dam toe near the crosshole set. A 50-lb electromagnetic surface vibrator was swept through frequencies of 30, 50, 70, 90, 120, and 150 Hz. The geophones were spaced at 2-ft intervals.

Test Results

53. The geophysical test results obtained at each location and the developed velocity profiles are described below.

Location 1:
Station 64+00L, Offset 2+40B

54. A suite of geophysical techniques was employed at Location 1 to establish V_s and V_p profiles in the downstream foundation area. Only surface techniques were at the dam crest, so foundation velocities were estimated

(by Ballard, 1978*) from the downstream results by adjusting for the increase in confining stress. Subsurface geophysical techniques were not used on the dam crest so as not to interfere with the railroad on the dam crest.

55. Surface refraction. Data collected from P-wave refraction seismic lines RS-1-P, RS-2-P, and RS-3-P near the downstream toe of the dam are presented as time versus distance plots in Figures 17 through 19, respectively. These plots indicate the presence of three V_p zones in the foundation. The first zone extends from 0 to about 20 ft and contains partially saturated soils with P-wave velocities ranging from 1,150 to 2,550 fps. The second zone extends from about 20 ft to bedrock, and has P-wave velocities that generally equal or exceed 4,800 fps, the V_p of water, indicating a high degree of saturation. The estimated depth to bedrock appears to have P-wave velocities ranging from about 12,500 to 19,000 fps.

56. Figure 20 shows the refraction results from RS-4-P performed on the crest of the dam. These results indicate the presence of two velocity zones within the embankment. The upper 5 ft of embankment shows a V_p of about 1,000 fps, underlain by material with a V_p of about 2,300 fps.

57. Figure 21 shows refracted S-wave data obtained from line RS-3-S, located along the toe of the dam. The data showed quite a range of results, possibly indicating a variation in the deposits across the 200-ft length of the refraction line. Each run indicated 2 zones. In one direction, the upper zone had an apparent V_s of 600 fps, the apparent interface depth was 13 ft, and the lower zone had an apparent V_s of 840 fps. In the other direction, the upper zone had a V_s of 410 fps, the apparent interface depth was 3 ft, and the lower zone had an apparent V_s of 690 fps.

58. Figure 22 shows refracted S-wave data obtained from line RS-4-S, located along the crest of the dam. The data indicate the presence of 2 zones. From 0 to about 7 ft, a V_s of 350 to 380 was measured. Below this depth, the material showed a V_s ranging from 720 to 780 fps.

59. Rayleigh wave. The surface vibrator test data were also examined with time versus distance plots for each frequency. The Rayleigh wave velocity for soil and rock is about 10 percent less than the corresponding shear wave velocity. The effective depth of investigation is approximately 1/2 the

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* Personal Communication, 1978, R. F. Ballard, US Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

wavelength of the propagation frequency. The estimated Rayleigh wave velocities from the field measurements are given in Table 6.

60. Downhole tests. The downhole work was performed at BEQ-2U, identified as borehole 1 in Figure 15. Data collected in the process of attempting to determine P-wave velocities appeared to have been overwhelmed by the presence of the casing and grout in the borehole. For this reason, the downhole P-wave data were not considered reliable and, consequently, were not used. A similar problem developed at the CPT sites described later in this section.

61. The downhole S-wave results, however, appear to be minimally affected by the casing and grout and are considered to be reliable. The time versus distance plot shown in Figure 23 indicates the presence of 5 velocity zones, listed in Table 7.

62. Crosshole tests. Figure 24 lists the V_s and V_p values, computed with CROSSHOLE from the crosshole field data, as a function of depth for each of the two receiver holes identified in Figure 15. As would be expected, data obtained between boreholes 1 and 3 exhibited slightly higher velocities than between boreholes 1 and 2 due to the greater spacing between boreholes 1 and 3 (28 ft as opposed to 20 ft). As borehole spacing increases, higher velocity layers tend to dominate, thus raising the average velocity between two points.

63. From examination of all the tests performed in this area, the V_s profile shown in Figure 25 was developed for use in the dynamic response calculations. For comparison, the downhole results and the SPT N-values and descriptive log from BEQ-3 and BEQ-6 are also shown in Figure 25. The S-wave signals could not be transmitted through the limestone bedrock at this location. It is assumed that the bedrock at this location is heavily eroded with solution cavities, as observed in the excavation for the powerhouse and in borings to rock. A V_s of 5,000 fps was assigned to the bedrock on the basis of WES experience with V_s measurements at other sites with cavernous limestone. As was observed at Location 3, a significant velocity inversion exists in Unit 2 of the foundation soils. Although nearby piezometers indicated water levels within a few feet of the ground surface, about Elevation 350 ft, the V_p profile indicates that soils in Unit 1 are not fully saturated to a depth of 20 ft. Below this depth, the V_p values generally exceed 4,800 fps, the V_p of water.

64. Estimated Centerline Profiles: Station 64+00L. The embankment centerline V_s and V_p profiles estimated from the RS-4 surface refraction tests and interpreted from the downstream foundation results are shown in Figure 26. The V_p field results indicate that the embankment and the top 20 ft of foundation material are not fully saturated. The presence of air bubbles of only 1 percent by volume is sufficient to reduce the V_p of a fully saturated soils from 4,800 fps to the 2,300-fps value exhibited by this zone. The foundation V_s values were estimated by (a) calculating K_2 and increasing the low-strain shear modulus, G_{max}, according to the increase in mean confining stress, σ'_m, due to the embankment, and (b) WES experience with similar materials and similar geometries. The K_2 values are computed from the following formula (see Seed et al., 1984, for typical K_2 values of a wide range of soil types):

$$G_{max} = 1,000 * K_2 * \left(\sigma'_m\right)^{1/2} \tag{1}$$

Where G_{max} and σ'_m are in psf.

Location 2:
Station 36+00L, Offset 0+39B

65. The velocity profiles computed from the S-wave and P-wave crosshole data obtained at this location are shown in Figure 27. The S-wave velocities increase from 500 fps to 940 fps over the depth interval 0 to 64 ft. From 64 ft to 118 ft, V_s decreases from 765 fps to 650 fps. The material from 118 ft to 130 ft (the bottom of the hole) exhibited a velocity of 900 fps.

66. The V_p profile is also shown in Figure 27. The P-wave velocities increased from 2,100 fps to 5,100 fps over the depth interval 0 to 56 ft. From 56 ft to 76 ft, V_p is about 3,500 fps, and below 76 ft, V_p ranges from 4,670 fps to 7,290 fps. The data indicate that materials at a depth of 50 ft and below are approximately fully saturated. The piezometers closest to crosshole set 2 were read on 3 April 1984, during the time period of the geophysical tests. The depth to water in piezometer BP-5 (midtip Elevation 301.4 ft in Unit 2), located about 80 ft to the right of crosshole set 2, read 49 ft, and the depth to water in piezometer BP-24 (midtip Elevation 274.5 ft in Unit 3), located about 50 ft to the left of crosshole set 2, read 53 ft. These piezometer readings agree well with the water level interpreted from the V_p results.

67. The borings used for the crosshole tests, WES 1-1 and WES 1-2, were installed by rotary drilling methods but no penetration testing or sampling was done. The log of the cuttings indicate the following: (a) compacted embankment clays were found to a depth of 38 ft, (b) the 2-ft thick drainage blanket was encountered from 36 to 38 ft, (c) the clay of Unit 1 was found from 38 to 57 ft, (d) the alluvial silts and sands of Unit 2 extended from 57 to 94 ft, (e) the gravelly, silty sand of Unit 3 was found from 94 to 127 ft, and (f) limestone bedrock was found from 127 ft to the bottom of the hole at 130 ft.

Location 3:
Station 34+45L, Offset 4+95B

68. The V_s and V_p profiles obtained for crosshole data at this location are shown in Figure 28. For comparison, the descriptive log and SPT N-values from nearby borings BEQ-7, BEQ-21, and BEQ-22, are also shown. The V_s profile shows a distinct velocity inversion in Unit 2 at this location. The lower velocity layer is also reflected in the reduced N-values at this depth. The V_s zones are 575 fps from 0 to 10 ft, 700 fps from 10 to 18 ft, 600 fps from 18 to 26 ft, and 475 fps from 26 to 42.5 ft. From 42.5 to 71 ft, the V_s varies from 560 to 680 fps. Below 71 ft, a V_s of 900 fps was observed. The V_p ranges from 2,225 to 3,340 fps from 0 to 27 ft. Below 27 ft, to the bottom of the hole, V_p exceeds 5,000 fps. Piezometers in this area indicate that a perched water table exists in Unit 1 and the water levels in Units 2 and 3 correspond closely with tailwater elevations. This same trend is indicated by the V_p results.

Locations 4 and 5: CPT 12
(Station 38+70L, Offset 2+07B) and
CPT 26 (Station 34+56L, Offset 4+98B)

69. The V_s and V_p profiles estimated from downhole tests with CPT equipment are shown in Figure 29 for CPT 12, located in the switchyard, and in Figure 30 for CPT 26, located just downstream of the switchyard near the tailrace canal slope. In the ERTEC report (Appendix F), it was noted that the P-wave results were affected by the CPT rods. The S-wave velocities are approximately 20 to 50 percent higher than those measured by crosshole methods. Since the crosshole method is considerably more reliable than downhole tests for accurate velocity measurement and layer definition, the CPT measured

velocity profiles were given very small weights in the development of velocity profiles for the switchyard area.

Estimated velocity
profiles for switchyard

70. To determine reasonable V_s and V_p zones for the switchyard, the K_2 values were computed from the crosshole test results at Locations 2 and 3. These K_2 values are shown in Figure 31 on a section of the dam that includes the switchyard. In the computation of K_2 values, it was assumed that level ground mean confining stresses with $K_o = 0.45$ was a sufficient approximation to the field stresses. The K_2 values from Locations 2 and 3 were averaged, to arrive at the K_2 values for the switchyard. Then V_s values were computed from the relationships between V_s, G, and K_2. As mentioned earlier, only minor consideration was given to the CPT results.

Summary

71. Geophysical measurements were made at study areas on the Right Embankment Dam at the Barkley Project. The types of tests included crosshole, downhole, and surface refraction to measure V_s and V_p. At one location at the toe of the dam, Rayleigh-wave tests were performed. From these tests, V_s and V_p profiles were developed for the dam centerline, the switchyard, and the downstream area. Usually the crosshole results provided the primary basis for the selected velocity zones used in the dynamic response calculations.

72. The shear wave velocity profiles were correlated with general site zones Units 1 thru 3. The results of this comparison are shown in Figure 32. This figure shows the complex site stratigraphy in that these zones do not distinquish themselves with particular S-wave velocities. The range is broad and similar for each unit. Further complications in correlating the data with these zones is that the data for Location 4 show higher velocities which is a function of the test method not the soil properties. Also, choice of a boundary between units at particular elevations also smears zones together as the site stratigraphy has been shown to be undulating. General interpretation of the zones is that the surface layers of the Unit 1 clays exhibit velocities in the range of 400 to 600 fps. Otherwise a velocity in the range of 700 to 800 fps is characteristic of Unit 1. Unit 2 composed of layered and mixed sands, silts, and clays shows a broad range of velocities as would be expected

from this type structure. Unit 2 velocities should be expected in the range of 450 to 950 fps. The soft clays accounting for low end of the spectrum and dense sands contributing the high end of this range. The velocities of Unit 3 again show a broad range from 550 to 1,025 fps. The lower velocities correspond to the silty, clayey sands and the velocities of 900 to 1,000 fps are characteristic of gravelly, dense sands, and the top of rock at the bottom of Unit 3. Choice of an average velocity for each unit would be misleading as it would not account for the complex stratigraphy which exists at this site. In conclusion, soft zones with a velocity 450 to 600 fps can be found in all units. Unit 2 is more populated with these soft zones. Dense sands and gravelly sands with velocities of 900 to 1,000 fps are more characteristic of Unit 3 but also exist in Unit 2.

PART VII: PIEZOMETERS

Locations and Readings

Locations

73. A total of 73 piezometers have been installed either before or during the seismic study. A plan view showing the locations of these piezometers is given in Figure 33, and Tables 8A through 8C give the midtip elevations. Below is a discussion of when, where, and why these piezometers were installed. A description of Units 1 through 3 of the foundation, referenced in the discussion below, can be found in paragraph 23.

- a. Existing. Prior to the beginning of the study in 1977, 25 piezometers had been installed at Barkley Dam (see Figure 33 for locations of piezometers BP 1-25). The purpose of these piezometers is to monitor the pore water pressures that exist in the dam and the foundation. Of these 25 piezometers, 2 have their midtips set in the embankment, 1 is located in Unit 1 of the foundation, 11 are set in Unit 2, and 11 are set in Unit 3.
- b. June 1979. To better define the groundwater regime, WES installed 6 piezometers near station 64+00L. These piezometers were set in two groups, with 3 piezometers per group. The midtips of the piezometers of each group were set at three elevation intervals (Elevations 330 through 326, 310 through 306, and 290 through 286, in feet).
- c. July-August 1979. In 1979, 5 additional piezometers were installed in the embankment next to the powerhouse for the purpose of monitoring possible excess seepage in this area. A sixth piezometer was installed through the crest of the dam at Station 49+71L with its midtip set in Unit 1.
- d. September 1981. Following the occurrence of a boil in the drainage ditch at the toe of the dam, 9 piezometers were installed between Stations 63+00L and 67+00L. The midtips of these piezometers were set at 2 general elevations, 5 at Elevation 340 ft, and 4 at Elevation 315 ft.
- e. 1982. After 8 SPT borings (BEQ 7-13) were drilled, piezometers were installed in the boreholes. All the midtips were set in Unit 3 of the foundation at about Elevation 285.
- f. 1984. Twenty piezometers were installed in the switchyard area, again after SPT borings (BEQ 15-34) were drilled. Nine of these had their midtips set in Unit 1, 10 were set in Unit 2, and 1 was set in the denser sands of Unit 3.

Piezometric levels

74. The piezometric levels and fluctuations in the embankment and foundation vary, depending on their location and midtip elevation and are

influenced by changes in the headwater and tailwater. As described in Part V: Pool Levels, the headwater normally varies only 4 ft throughout the year but the tailwater can vary as much as 20 ft or more. Tables 8D through 8H give typical piezometric data for two dates (6 March and 2 July 1985) which reflect the annual extremes of the pool levels and show the extent that these piezometric levels are influenced by headwater and tailwater. Piezometer readings for 1984-1985 are shown in Appendix A. These levels and data are discussed below for each of the units of the foundation.

a. Unit 1. Piezometers in this unit show relatively flat piezometric levels throughout most of the year, fluctuating only a few feet or less. Those located along the main part of the embankment have water elevations between 340 and 350 ft and appear to be influenced by the headwater as shown by Table 8D. Piezometers BP-9, 15 and BD-1, 2 and 5 appear to lag the headwater while the remaining piezometers show a more direct response to headwater and may be influenced by a sand layer in this unit that was discovered when a boil occurred in this area in 1981. The piezometers in the switchyard also have relatively flat piezometric levels but generally range between Elevation 330 and 340 ft and, like those located along the main part of the embankment, fluctuate little throughout the year. These piezometers also appear to react with headwater but with a lag time.

b. Unit 2. The piezometric levels along the main part of the dam are relatively stable throughout the year with water elevations ranging from 340 to 348 ft and fluctuations of less than several feet (see Table 8E). Figure 34 is a profile of the dam showing the piezometric levels for readings taken on 6 March and 2 July 1985. Headwater appears to be influencing the piezometers along the main embankment while the tailwater influences the piezometers in the switchyard (see Table 8F). A seepage gradient toward the river is also evident and is steep between piezometers BEQ-23 and BEQ-25 as indicated by the contours of the water levels shown in Figures 35 and 36.

c. Unit 3. In the denser sands and gravels of this unit the piezometers generally react to the tailwater fluctuations as shown on the profile along the embankment in Figure 37 and summarized in Table 8D. As the distance from the tailwater increases the fluctuations and reactions to changes in the tailwater are not as great.

d. WES Piezometers. This series of piezometers was installed to help define the groundwater regime and water levels, and as can be seen by Table 8E, a downward seepage gradient exists.

Summary

75. For most of the length of the dam the groundwater levels are close to the ground surface, generally between Elevation 340 and 350 ft with fluctuations of only a few feet. In the switchyard area, the tailwater influences the piezometric levels with fluctuations of 20 ft or more over the course of the year not unusual. A normal seepage gradient exists near the tailrace, but a downward seepage gradient also exists as shown by the series of WES piezometers.

PART VIII: SPT INVESTIGATION RESULTS

General

76. The geology of the river valley and the construction records indicated that the foundation soils beneath the dam were deposited in a complex, repeating sequence, consisting of deposition in a series of layers, erosion by the course of the river, followed by further deposition. Profiling and characterizing such a complex layered system for liquefaction potential evaluation and seismic stability assessment was a difficult task. The field investigations were carried out in several episodes to develop an understanding of the locations, thicknesses, and areal extent of potential problem zones. The field work was conducted in the downstream area, and it was assumed that the upstream condition would be adequately represented by the downstream observations. In the course of the investigations, 44 SPT borings and 11 undisturbed borings were drilled. A brief description of when and where they were drilled is given below. Figure 38 is a plan view showing the location of these borings. Table 9 gives the location, depth, and other pertinent information for the SPT borings. The undisturbed borings are discussed in Part X.

a. 1977. The initial exploration program consisted of 5 SPT borings, BEQ-1 through BEQ-5, spaced along the downstream toe of the dam on about 1,000-ft centers, all drilled to rock. The drilling was performed with a trip hammer by WES. After a review of the boring logs, it was decided that the area in the vicinity of BEQ-3, Station 64-00L, would be a typical representation of the foundation. An additional SPT, BEQ-6, and two undisturbed borings, BEQ-1U and BEQ-2U, were drilled in this area. As discussed in Part VI, geophysical measurements were also made in this area soon after the boreholes were drilled and cased.

b. 1979. After review of the initial data, it was decided that additional borings would be required. WES drilled two undisturbed borings, DS-1 and DS-2, in the same general area as BEQ-1U and BEQ-2U. SPT's were conducted in DS-1 and DS-2 at depths where gravels were encountered. A third boring, DS-3, was drilled close to the river bank at Station 34+28L, 4+81D. This boring was made by alternating SPT and undisturbed samples. All SPT measurements were made with a trip hammer. Six piezometers were installed in the vicinity of Station 64+00L to determine if a downward seepage gradient existed. The piezometers were divided into two groups of three set at elevation intervals of 330 through 326, 310 through 306, and 290 through 286 ft.

c. 1981. During the summer of 1981, the drainage ditch at the toe of the dam was cleaned out. This resulted in the occurrence of a sand boil. To determine the areal extent of this sand layer, ORNED drilled 9 SPT borings, BD-1 through BD-9, with a rope and cathead system and a safety hammer. All the borings were drilled to about Elevation 310 ft, and the data was included in the SPT database.

d. 1982. In December 1982 and January 1983, ORNED drilled 8 additional SPT's, BEQ-7 through BEQ-14, with rope and cathead equipment and a safety hammer for the purpose of correlating the sand and clay layers in the foundation and for adding more SPT data to the data base. These borings were split-spaced between BEQ-1 through BEQ-5, except for BEQ-10 and BEQ-13 which were drilled next to BEQ-2 and BEQ-4, respectively, to obtain data and samples lost from the initial drilling.

e. 1984. To perform a more detailed investigation in the vicinity of the switchyard, ORNED drilled 20 SPT's with a rope and cathead system and a safety hammer, BEQ-15 through BEQ-34, in this area during the months of May through September, 1984. Six undisturbed borings, BEQ-3U through BEQ-8U, were drilled to obtain samples for laboratory testing of the embankment dam and switchyard fill, clays from foundation Units 1 and 2, and sands from the foundation. Borings BEQ-5U, BEQ-6U, and BEQ-8U were specifically drilled to obtain samples of the foundation sands for determination of in situ steady-state strengths.

77. In this part of the report, the procedures used for each of the SPT drilling efforts will be described. The SPT results include blowcounts, jar samples from each split-spoon sample, and the results of index tests. The SPT results were organized and stored in a data base for liquefaction potential evaluations and to assist with site characterization. These results and the corresponding data base will be presented in this chapter.

SPT Field Investigation Procedures

78. The 1977 and 1979 field work was performed by WES. In the WES procedure, a 140-lb hammer is dropped 30 in. with a trip hammer to drive the split spoon through the first 18 in. of the sequence, and the hole is then advanced another 18 in. for a total depth of 3 ft, with a modified fishtail bit (Goode, 1950). This fishtail bit has been modified with baffles which deflect the drilling mud in an upward direction. The hole is uncased and filled with drilling mud. The liner is omitted from the splitspoon sampler, and the type of rods used are "N" rods. It is estimated that the energy ratio for the trip hammer is about 80 percent. To determine equivalent SPT N-values

for a rope and cathead system (an energy ratio of 60 percent), the trip hammer blowcounts need to be multiplied by a factor of 1.3 (Seed et al., 1984).

79. One difficulty in the procedure is that there is an 18-in. blind spot in the boring log that is larger than the typical thickness of soil layers in this intensely-stratified deposit. This blind spot complicates attempts to correlate layers between borings. However, cleanout distances between SPT drives of less than 1 ft may lead to disturbance of layers immediately in front of the advancing split spoon, and consequently misleadingly lower blowcounts.

80. The remaining SPT work for this study was performed by ORNED with a rope and cathead system. ORNED performed SPT's with two different cleanout distances. Continuous SPT's refer to borings with no cleanout distance, so a continuous observation can be made of the underlying soil layers. In a continuous SPT boring, a modified fishtail bit is used to clean out the hole only through the same depths that the split spoon was driven. Standard SPT's refer to borings with a cleanout distance between split-spoon drives of 1 ft or greater. A column in Table 9 indicates the method of drilling for each of the SPT borings.

Laboratory Index Testing

81. Index property tests were performed on nearly all of the SPT samples in accordance with EM 1110-2-1906. In general, the tests performed were natural water content, Atterberg limits, sieve analysis, and hydrometer. If enough material was available, these tests were performed on almost all samples for BEQ-1 through BEQ-14. For samples BEQ-15 through BEQ-34, the above tests were performed under the following guidelines: (a) Perform natural water content on all samples, (b) If the liquid limit is greater than 35 or the water content is less than 0.9 times the liquid limit, then do not perform sieve or hydrometer, (c) If percent passing the No. 200 sieve is less than 5, do not perform hydrometer. These guidelines result from the criteria needed to assess liquefaction resistance of soils containing fines.

82. Field personnel logging the material were instructed to save the entire 18-in. drive and take separate jar samples for each type of material. Because of the interbedded nature of the foundation, this was difficult to accomplish; consequently, many of the jar samples were mixtures. Laboratory

personnel were instructed to separate the different layers, if possible, and perform the above tests in the separated samples if enough material was available. The uncertainty in the index tests was qualitatively appreciated, but, it is beyond the state of the art at the time of this writing to quantitatively assess the effect of the fine-grained, stratified soil fabric on the cyclic strength of the soil based on the results of SPT N-values.

SPT Data Base of Field and Laboratory Results

83. An enormous quantity of data was compiled from the SPT field and laboratory work to characterize the site and assess the liquefaction resistance of the foundation soils. The following information is needed for analyzing individual blowcounts: the exact location and top-of-hole elevation of the boring it comes from, the water level at the time of sampling, the unit weights of the overlying soils, the depth interval of the SPT drive, the drilling method (i.e., trip hammer or rope and cathead), the blowcounts for each 6 in. of the 18-in. drive, sampling losses, identification of each jar sample taken from the split-spoon for the drive, field classifications, and laboratory index test results for each soil layer from the jar samples. The index test data recorded for each laboratory sample was: the grain size distribution in terms of D60, D50, D30, D10, percent passing the No. 200 sieve, and percent finer than 0.005 millimeters; the Liquid Limit, LL; the Plastic Limit, PL; and the natural water content, Wn.

84. All the SPT field and laboratory data are stored in 3 data bases, which are printed in their entirety in Appendix B. The first data base is the boring data base which identifies the name, location and other pertinent details for each SPT boring. The second data base is the SPT sampler data base, which identifies the locations, depths, blowcounts, and number of jar samples for each SPT drive. The third data base is the laboratory index test data base which identifies the location and depth interval for each jar sample, and the results of the laboratory index tests. The fields in each of the data bases are described in more detail below.

SPT boring data base

85. The SPT boring data base is the shortest of the three, and is shown in Table 10, as well as in Appendix B. The individual fields are:

a. "Boring Name" - This column identifies the SPT boring by name. BEQ-20 is an example.

b. "Ground Elevation" - This field gives the top-of-hole elevation for the boring in ft with mean sea level as the datum. For example, BEQ-20 has a top-of hole elevation of 364.70 ft.

c. "Location East" - This field gives the Station for the boring as measured along the axis of the right embankment, which runs east-west. For example, boring BEQ-20 is located at Station 39+85L and the entry in the data base is "3985."

d. "Location North" - This column give the offset from the dam axis. All offsets are downstream, in the north-south direction. For example, BEQ-20 is located at Offset 1+70B and the column entry is "170."

e. "Drilling Method" - This field identifies the type of drill rig used to perform the SPT boring. For example, BEQ-20 was drilled with Mobile B-53 equipment.

f. "Depth Maximum" - This field records the maximum depth of the boring in ft. For example, the maximum depth of boring BEQ-20 is 81.00 ft.

g. "Water Table Depth" - This field records the best estimate of the depth to the water table in ft at the time the boring was made. The depth to the water table in boring BEQ-20 was estimated to be 24.70 ft.

SPT sampler data base

86. The SPT sampler data base identifies the boring name and depth interval for each SPT drive, and records the number of blows for every 6 in. of the drive. The data for each boring are started on a new page. The jar samples recovered from each drive are numbered. The complete SPT sampler data base is given in Appendix A. One page from the data base is shown in Table 11, and corresponds to the samples retrieved from boring BEQ-20. The individual fields are:

a. "Boring" - This column identifies the boring that the sample comes from. This column can be keyed to the boring data base for any other information located there such as station and offset. Boring BEQ-20 will continue to be used as an example.

b. "Sampler Top" - This field records the depth in ft at the top of the SPT drive. This field was used to identify individual SPT drives.

c. "Sampler Bottom" - This field records the depth in ft at the bottom of the SPT drive.

d. "0-6" - This column gives the number of blows for the first 6 in. of the SPT drive.

e. "6-12" - This field give the number of blows for the second 6 in. of the SPT drive.

f. "12-18" - This field gives the number of blows for the last 6 in. of the SPT drive.

g. "Number of Samples" - This column indicated the total number of samples taken from the split-spoon and subjected to index tests in the laboratory.

h. "Sample 1, 2, 3, 4, 5, 6" - These columns identify each of the laboratory samples taken from the SPT drive. The jar samples are numbered sequentially, but if the laboratory personnel decided that more than one type of soil was present in the jar, they divided the jar sample into up to 6 individual samples. Separate samples from the same jar are identified by a letter after the jar number. For example, for boring BEQ-20, the SPT drive beginning at a depth of 61.5 ft had three jar samples taken from the split spoon, numbered 097, 098, and 099. Jar samples 097 and 098 were divided into 2 separate soil samples in the laboratory for index testing, resulting in a total of 5 soil samples for this one SPT drive. The 5 samples are numbered 097A, 097B, 098A, 098B, and 099.

87. The boring name links the SPT sampler data base with the boring location data base. By this means, the exact location of each SPT drive is uniquely stored and accessible for further analysis.

SPT laboratory test results data base

88. This data base stores all the laboratory index test results for the many jar samples and subdivided jar samples obtained in the SPT field work. The boring name and laboratory soil sample number (and letter, as appropriate) link this data base with the preceding two data bases described above. The test results data base stores the key information needed to address the liquefaction susceptibility criteria. A one-page printout from the SPT laboratory test results data base for boring BEQ-20 is shown in Table 12. The data base is organized as follows:

a. "Boring Number" - This field identifies the SPT boring the sample comes from. An example is BEQ-20.

b. "Sampler No." - This field gives the identification of the SPT soil sample tested in the laboratory, from the list of samples in the SPT sampler data base described previously. For example, sample No. 097A from boring BEQ-20.

c. "Top Sample" - This column lists the beginning of the depth interval in feet from which the soil sample is taken. For example, soil sample No. 097A in boring BEQ-20 starts at a depth of 61.50 ft.

d. "Bottom Sample" - This column lists the lower end of the depth interval in feet from which the soil sample is taken. For example, soil sample No. 097A, boring BEQ-20, is a sample of soil taken from the depth interval 61.5 ft (from the previous column) to 61.6 ft. (from this column).

e. "Natural water (Wn)" - This field gives the natural water content (in percent) of the soil sample. To continue with the example, sample No. 097A from boring BEQ-20 has a natural water content of 26.20 percent.

f. "Liquid Limit (LL)" - This field lists the liquid limit (in percent) of the soil sample. No Atterberg limits were determined for the example sample No. 097A. The column entry to indicate no data is zero.

g. "Plastic Limit (PL)" - This field lists the plastic limit (in percent) of the soil sample. The entry for the example sample No. 097A is zero indicating, in this case, no limit tests were performed.

h. "D 60" - This column and the next four columns list results from sieve analyses. Sixty percent (by weight) of the soil sample is finer than this grain size diameter (given in millimeters). The D 60 value for the example sample No. 097A is 0.040 mm.

i. "D 50" - Fifty percent (by weight) of the soil sample is finer than this grain size diameter (given in millimeters). Sample No. 097A has a D 50 of 0.025 mm.

j. "D 30" - Thirty percent (by weight) of the soil sample is finer than this grain size diameter (given in millimeters). Sample No. 097A has a D 30 of 0.008 mm.

k. "D 10" - Ten percent (by weight) of the soil sample is finer than this grain size diameter (given in millimeters). The D 10 for sample No. 097A is too fine (less than 0.005 mm), so there is no datum. The entry "-1.0" indicates no datum.

l. "Percent Pass No. 200" - This column give the percent (by weight) of the soil sample that is finer than the No. 200 sieve. Sample No. 097A has 70.1 percent by weight passing the No. 200 sieve size.

m. "Percent Pass No. .005" - This column gives the percent (by weight) of the soil sample that has particle diameters less than 0.005 millimeters as measured in a hydrometer test. Sample No. 097A has 23.0 percent by weight finer than 0.005 mm.

n. "Word Classification (Minor)" - This field lists adjectives to the major word classification for the soil sample. The Unified Soil Classification System (USCS) is used. The modifier for sample No. 097A is SANDY.

o. "Word Classification (Major)" - This field lists the major word classification for the soil sample. The word classification for sample No. 097A is CLAY.

p. "USCS Soil Class" - This field lists the symbol for the USCS classification of the soil sample. Sample No. 097A classifies as CL.

q. "Color (Minor)" - This column gives a modifier for the color description of the soil sample. The modifier for sample No. 097A is DARK.

r. "Color (Major)" - This column gives the overall color description of the soil sample. The major color description of sample No. 097A is GRAY.

89. The 3 data bases for this field and laboratory work required over 700,000 bytes of hard disk storage. There are 44 boring entries (one entry as used here refers to a full line of data in the data base), 1424 SPT sampler entries, and 1869 soil sample entries. In addition to the data base printouts shown in Appendix A, diskettes are provided.

Field and Laboratory Data Plots

90. Information from the data bases were plotted in various forms as necessary to characterize the foundation and assess liquefaction potential. Special software was developed specifically for this purpose. Appendix B shows plots of the data versus depth for each hole. These figures show the following information plotted versus depth: measured SPT blowcount, mean grain size (range and weighted average), percent smaller than 0.005 mm (range and weighted average), percent passing the No. 200 sieve (range and weighted average), natural water content and plastic and liquid limits. Some assumptions were made concerning missing samples, and the logic for dealing with this lack of data in the generation of the plots is described in detail in Appendix B.

PART IX: STREAMBANK EXCAVATION

General

91. Preliminary liquefaction analyses described in Volume 4 of this series indicated that Unit 2 of the alluvial foundation soils needed further investigation. As the SPT and undisturbed sampling investigations (described in Parts VIII and X, respectively) progressed, it became evident that it was usually not possible to correlate individual soil layers observed in one boring with those observed in another boring located only 10 ft away, in a direction parallel to the axis of the dam. The high degree of stratification and lack of horizontal continuity in the direction of the dam axis made detailed mapping of the soil profile difficult at best. From geological reasoning, it was expected that more continuity of soil layers should exist in the direction of river flow, perpendicular to the axis of the dam. Photographs of these soil layers in the direction of river flow can be found in Appendix C. Continuity of layers was important to establish because of the implications for slope stability. Potentially liquefiable soil layers of extended length and width have significantly more impact on slope stability than similar layers with relatively limited areal extent.

92. Up to this point, the field investigations did not examine continuity in the direction of flow. Due to the uncertainty that would remain from borehole correlations and the uncertainty and lack of adequate resolution from geophysical methods, visual examination of soil layer continuity was determined to be essential. Since the foundation materials in question were at some depth, roughly 15 to 55 ft, deep test pits or trenches would be very expensive. It was determined that examination of exposures of Unit 2 in the streambanks downstream of the dam would be a more practical solution since it would provide the necessary information at a much lower cost (photos of the soil layers in the direction of river flow can be found in Appendix C). A reconnaissance was made of the riverbanks downstream of the dam for a distance of about 3 miles. An exposure of materials considered to be representative of those of concern in Unit 2 was found about 1.5 miles downstream of the dam on the right bank. See attached location map, Figure 39. This exposure was developed and mapped during the period 31 October to 1 November, 1983.

Field Procedures

93. Although the bank exposure was most suitable for mapping, work was needed to enlarge it and to clean it up for detailed logging and photographing. Shovels and an entrenching tool were used to enlarge the natural faces while a garden hoe and a wide blade putty knife were used to shave the soil face to give a good, clean surface for logging and photographing. When this was done, stationing stakes were set out for use as horizontal reference points during the logging operation. For vertical control, a string was fastened to two stakes and the string line set horizontal at a known elevation. This line would be reset at different elevations as needed. All logging measurements were made from these reference stakes and elevation lines. The elevation of the river, which was determined from data obtained from the Barkley Project, was used as a starting elevation. The needed elevations for logging purposes were hand leveled in from the river's edge located just a few feet away. Since two geologists were present, one made all the measurements and described the materials while the other sketched in the soil faces on a scaled drawing of the exposure and recorded all information.

Results

94. The soil faces that were mapped were oriented parallel to the river, so nothing was learned of the nature of the soil beds in a direction perpendicular to the river. The final dimensions of the mapped exposure were about 30 ft long by 5.5 to 6 ft high. The maximum thickness of an individual soil layer was 1.5 ft. The average thickness of the beds would be on the order of about 2 to 4 in. and were generally undulating in nature. Lengths of beds varied greatly, from several inches to lengths greater than the mapped exposure (30 ft). One bed outside the limits of the exposure was traced for a distance of about 150 ft before it could no longer be traced. The geologic section that was developed from this mapping is shown in Figure 39. It should be noted that some generalizations in descriptions had to be made during the logging. A bed shown in Figure 39 may contain minor lenses or zones of material that may vary in description from what was logged. Based on this field exercise, it was concluded that significant continuity may exist in soil layers in the direction parallel to the river, and this assumption was employed in subsequent stages of the seismic stability evaluation.

PART X: UNDISTURBED SAMPLING AND LABORATORY RESULTS

General

95. As discussed in Part VIII, field drilling and sampling was performed at various times during the seismic safety study. Undisturbed samples were used to estimate in situ density, to observe foundation stratigraphy in detail, and to perform undrained laboratory strength tests with both cyclic and monotonic loading. Three excursions were made to obtain undisturbed samples, the first in 1977, the second in 1979, and the third in 1984. Table 13 identifies the undisturbed borings and Figure 38 shows their locations. The 1977 and 1979 efforts were performed by WES and were directed primarily toward obtaining foundation samples for undrained monotonic and cyclic testing. The 1984 field work was performed by the Nashville District, and was directed primarily toward obtaining undisturbed samples of: (a) the embankment for undrained monotonic testing, and (b) the foundation for steady-state strength testing. The field procedures and equipment used and the laboratory tests performed will be described for each of the undisturbed sampling efforts.

1977 and 1979 Field and Laboratory Studies

Field procedures

96. In the 1977 field work, undisturbed borings BEQ-1U and BEQ-2U were drilled in the vicinity of Station 64+00L. In 1979, 3 more undisturbed borings were drilled: DS-1 and DS-2 near Station 64+00L, and DS-3, in which undisturbed sampling was alternated with SPT sampling, near the river bank (Station 34+28L, 4+81D). The sampling sequence consisted of a 2.4-ft continuous push of the sampler followed by a 0.6-ft advance of the hole with a WES-modified fishtail bit. As discussed in Part VIII, the SPT's were conducted using a trip hammer in the first 18 in. of the sequence, then the hole was advanced another 18 in. with a modified fishtail bit.

97. The undisturbed samples were obtained with a 3-in. diameter Hvorslev Fixed Piston Sampler and drilling mud. When gravel was found in the foundation borings BEQ-1U and BEQ-2U, a fixed piston tube sample was difficult or impossible to obtain. In this case, the WES drillers used a Pitcher sampler. It was necessary to use the Pitcher sampler in one or both of borings

BEQ-1U and BEQ-2U in the following depth intervals: 73-75 ft, 82-90 ft, and 102-114 ft. SPT's were conducted in borings DS-1 and DS-2 where gravels were encountered.

Visual boring logs

98. Index testing of undisturbed samples was not complete except for the laboratory specimens, and several discrepancies were found between the visual field classifications and ultimate laboratory classifications. Typically, soils classified as silts in the field usually turned out to be clays. Consequently, the field logs of these holes were used only in a limited, qualitative manner. An appreciation of the complex layering of the foundation soils was emphasized by examination of the untested samples from this field work, which were split open for visual study. Photographs of these samples are shown in Appendix G.

Laboratory testing

99. Upon arrival at the WES laboratory, the foundation samples were placed in a freezer to minimize further disturbance during sample handling. The foundation soils were divided into three groups: sands, nonplastic silty sands, and specimens with plastic fines. The laboratory index tests included density, specific gravity, mechanical analysis, maximum and minimum density, and Atterberg limits. The triaxial tests consisted of isotropically consolidated, undrained, stress-controlled, cyclic triaxial tests (CTX) and isotropically consolidated, undrained, stress-controlled compression shear tests with pore pressure measurements ($\bar{R}$). The CTX tests were meant to determine cyclic strength and the $\bar{R}$ tests were performed to study the dilative and contractive behavior of the soils at various void ratios and confining stresses. It was later decided that freezing samples with such high fines contents significantly altered the undrained monotonic and cyclic strength, so only brief mention is made of these test results.

100. Both undisturbed and reconstructed specimens were tested. Composite material representative of each soil group was obtained by combining appropriate material from undisturbed samples for the laboratory-compacted specimens. Laboratory tests were conducted in accordance with EM 1110-2-1906. The characteristics of the composite materials are presented in Table 14. A total of 20 $\bar{R}$ tests were performed, and individual test details are summarized in Table 15. The test numbers indicate the boring and depth of the sample. For example, test number 2-51.7 comes from boring DS-2 at a depth of 51.7 ft.

The letter "R" or a dual depth indicates a remolded specimen. As mentioned earlier, the CTX results were not used to determine cyclic strength in situ. The test program and results are given in Table 16 for general information purposes only.

1984 Field and Laboratory Studies

Field procedures

101. In the 1984 field work, 6 undisturbed borings were drilled by the Nashville District (ORNED) in the vicinity of the switchyard. Table 13 identifies the borings and Figure 38 shows their locations. A 3-in. diameter Hvorslev Fixed Piston Sampler was used and the samples were handled in a manner similar to the 1977 and 1979 field work. Borings BEQ-3U, BEQ-4U, and BEQ-7U were drilled to obtain samples of the embankment and switchyard compacted filled and the clays of foundation Units 1 and 2 for laboratory determination of undrained and effective shear strengths. These tests were conducted at the South Atlantic Division Laboratory (SAD) and at the Ohio River Division Laboratory (ORD). Logs of the samples and laboratory test data sheets are given in Appendix H.

102. Borings BEQ-5U, BEQ-6U, and BEQ-8U were drilled specifically to obtain high quality samples of the sand layers in the foundation for steady-state strength testing. A representative of Geotechnical Engineers, Inc. (GEI) was present to make key measurements during the sampling operations to quantify the void ratio changes in the soils due to the sampling process. The samples were transported by the GEI representative for steady-state testing at the GEI laboratory in Winchester, Massachusetts. Detailed description of this work, including the boring logs, and the laboratory test data, are given in the GEI report in Appendix D.

Laboratory testing for embankment and switchyard fill and foundation clays from Units 1 and 2

103. The objective of these tests was to determine effective and consolidated-undrained shear strengths of the embankment and switchyard fill and of the clays of foundation Units 1 and 2. A total of thirty-two isotropically consolidated, strain-controlled, undrained triaxial shear compression tests with pore pressure measurements ($\bar{R}$) were performed on samples from

BEQ-3U, BEQ-4U, BEQ-5U, and BEQ-7U. Four similar tests, but without pore pressure measurements (R), were performed on samples from BEQ-4U and BEQ-5U. Additional testing included Atterberg limits, sieve and hydrometer analyses, laboratory vane shear, and pocket penetrometer tests. The laboratory test data sheets are given in Appendix H. Table 17 summarizes the strength and index test results, and Table 18 shows the results of the pocket penetrometer and laboratory vane shear tests. The embankment and switchyard fill has a peak effective friction angle of about 32 degrees, and the clays of foundation units 1 and 2 have a peak effective friction angle of about 33 degrees. A sample of wood was found in BEQ-7U at an elevation of about 300 ft. Carbon dating revealed the sample of wood had an age of about 10,000 years.

Steady-state strengths of foundation sands

104. GEI performed 13 isotropically consolidated, strain-controlled, compression-loading, undrained triaxial tests with pore pressure measurements ($\bar{R}$) on undisturbed specimens of sand layers from the Barkley Dam foundation. Due to the intense layering of the sand and clay layers, the undisturbed specimens that could be tested had a length-to-diameter ratio of 1.3 to 1.6. Thus it was necessary to use lubricated end platens to minimize end friction. X-ray photographs of the tube samples were used in identifying appropriate test specimens before cutting the tubes. Seven laboratory vane shear tests were performed on samples of silty clay and sandy clay which were adjacent to the undisturbed $\bar{R}$ test specimens.

105. According to the steady-state theory developed by GEI, the undrained, steady-state shear strength of the foundation sand is a function only of the void ratio in situ. Table 19 and Figure 40 show the steady-state shear strengths and void ratios measured in the laboratory and estimates of appropriate values for the foundation sand. Because of the unavoidable densification during sampling and consolidation, the as-sheared void ratio is lower than the in situ value, resulting in a measured laboratory strength which is higher than the actual in situ strength. Therefore, the in situ steady-state strengths were estimated by correcting the measured $\bar{R}$ strengths to account for the difference between the in situ and as-sheared void ratios. The in situ void ratios were estimated by correcting the measured laboratory void ratio to account for all the changes in sample density which occurred during sampling, transport, handling, tube cutting, extrusion, and consolidation. The

estimated in situ steady-state shear strengths of the sand layers range from 5 to 94 psi. The recommended value for safety evaluation is 8 psi. Only two of the eleven estimated in situ strengths fall below this value (R-10 at 6 psi and R-12 at 5 psi), and this recommended value is approximately the average of the four lowest strengths. The results of these strength tests are shown in Figure 40.

PART XI: CPT INVESTIGATION RESULTS

General

106. CPT field investigation techniques were used to reveal stratigraphy and measure in situ strength because they have advantages particularly important to the Barkley site: the technique provides a continuous record, can resolve stratigraphic changes with a resolution of a few inches, and has a relatively low cost per foot. In the course of the seismic safety evaluation, it was determined that the switchyard-riverbank area was a critical zone. CPT investigations were performed only in this area. In later stages of the study, the CPT results were used qualitatively for stratigraphic correlation to estimate continuity and areal extent of problem zones, and quantitatively for liquefaction resistance and after-earthquake strength determination.

107. In the first phases of site investigation, 1977 to 1979, an effort was made to experiment with the Wissa piezometer probe as a tool for assessing the dilative or contractive behavior of the soils below the water table. Ardaman & Associates performed five piezometer cone probings at the dam site, three near Station 64+001, and two near the riverbank. Their report is given in Appendix I. These data play a very minor role in the site characterization effort.

108. Sixty-five CPT soundings were performed in the switchyard and riverbank area by Geoelectronics and the Earth Technology Corporation (ERTEC) during the period 15 through 27 May 1985. Thirty-four of these soundings included electrical conductivity measurements, thirteen soundings included piezometric measurements, and two soundings included downhole seismic velocity measurements. Separate dielectric probe soundings were performed to measure soil dielectric properties at two locations. A summary of the 1985 testing program is provided in Table 20, and Figure 41 is a plan view of the layout of CPT investigations. For a complete description of the many types of probings performed, see the ERTEC report in Appendix F. The discussions in this part are limited to the standard cone and sleeve resistance measurements.

109. This chapter describes the planning of the CPT investigation, types of tests selected, equipment used, field procedures, quality assurance precautions, results, and data base for storage and manipulation of the

results. The Wissa probe results were used in an approximate, qualitative manner, and only brief mention will be made of these results.

Wissa Probe Soundings

110. In 1978, Ardaman & Associates, Inc. of Orlando, Florida, conducted five Wissa probe soundings in the foundation soils at Barkley Dam to determine whether the soils below the water table contract or dilate during shear. Appendix I contains the complete Ardaman & Associates, Inc. report of the Wissa Probe soundings of Barkley Dam. Three of the probings were located on a line parallel to the toe of the dam at Offset 2+41B by taping a 10-ft distance from undisturbed boring DS-2 (Station 63+60L, Offset 2+31D), and were spaced 20 ft apart. The two other piezometer soundings were located in the vicinity of boring DS-3 (Station 34+28L, Offset 4+81D), and are shown in Figure 41. Piezometer probe readings were recorded below the water table to a maximum depth of 80.5 ft. The following conclusions were drawn from these soundings:

- a. The soundings indicate that very loose foundation soils are present, especially in the vicinity of Station 64+00. The loose materials appear to be localized pockets or lenses of limited areal extent and are not more than a few feet thick.
- b. There is a downward seepage gradient between Elevation 330 and 370 ft (MSL) at Station 64+00. Loose materials were encountered in this depth interval.
- c. Little reliable information was obtained in the area of Station 34+00, near the tailrace canal slope.

Planning CPT Locations for ERTEC Soundings

111. The CPT soundings had two objectives, to reveal stratigraphy and to estimate strength. To study stratigraphy, long strings of closely spaced soundings were made parallel and perpendicular to the dam axis through the switchyard and downstreams of the switchyard, as shown in Figure 41. In the streambank excavation described in Part IX, foundation layers were found to extend for distances of 5 to more than 30 ft in the direction of flow, and thicknesses of the layers also varies. Continuous layers with lengths of 30 ft or greater have a more significant effect on stability than smaller, discontinuous layers. A spacing of about 25 ft between probings was estimated to be a practical limit for layers that extended over lengths of about 30 ft

or greater to be detected by at least 2 soundings. The spacing of most of the probes was about 40 ft. CPT strings parallel to the dam axis are best for detailing valley stratigraphy such as identifying channel cuts and sandbar locations and determining the variation in soil strengths along these surfaces.

112. To relate strengths and stratigraphy determined from CPT results to other observations in the vicinity of the switchyard, CPT soundings were positioned near SPT and undisturbed sampling borings, as shown in Figure 41.

Selection of CPT Equipment

113. A standard CPT test involves pushing a 1.4-in. diameter probe into the earth at a rate of 2 cm/sec while monitoring the cone or tip resistance, q_c , and the sleeve friction resistance, f_s . The cone resistance is a bearing capacity measurement of the cone tip. The sleeve friction is a localized strength measurement of the soil as it passes a cylindrical steel sleeve located just behind the cone tip. These simultaneous measurements are made by means of electrical strain gauges bonded inside the probe unit. Continuous electric signals are transmitted by a cable in the hollow sounding rods to electrical equipment in the CPT truck. Cone and sleeve friction resistances are recorded versus depth in both analog and digital form. A set of hydraulic rams are used to push the cone and rods into the earth. The Earth Technology Corporation used a specially designed, all-terrain drive, 23-ton, heavy-duty truck to transport and house the CPT equipment.

114. Two different types of cone instruments, a subtracting cone and a tension cone, were used during this study to assure accuracy within the limitation of the equipment. The subtracting cone has a high-strength capacity, but requires careful calibration and may not accurately measure sleeve friction in low-strength materials without careful calibration and equipment warm-up. The tension cone measures sleeve friction accurately in low-strength materials, but can be damaged in a high-stress push, such as penetration of dense sands and gravels. The SPT and undisturbed sampling borings show that the gravel zones are present in the foundation. By using the subtraction cone in most soundings, following careful quality assurance procedures in the field, and using the tension cone where there was little danger of damaging the probe, it was possible to obtain accurate, high-quality q_c and f_s

measurements. The subtraction cone was used for the majority of the testing program. The tension cone was used for soundings CPT-8, CPT-25, CPT-30, CPT-32, CPT-43, CPT-55, and CPT-58. Tension and subtraction cone sleeve friction readings compared very well. Companion soundings were made as close as 10 to 15 ft apart. Each of the cone types is described in more detail below.

Subtraction cone

115. The subtraction cone consists of a conical tip with a 60-degree apex angle and projected cross-sectional area of 15 square cm, and a cylindrical friction sleeve with a surface area of 200 square cm. The tip, sleeve, and rods have outer diameters of 4.37 cm. A diagram of the subtraction cone is shown in Appendix F. The subtraction cone is a robust design with over 20-ton push capacity. There are two sets of strain gauges; one set measures cone-tip force and the other measures the sum of cone-tip force and sleeve friction. The sleeve friction, f_s, is the difference between these measurements. If f_s is less than 10 to 30 lb, then it cannot be resolved due to calibration and zero drift errors of the system.

Tension cone

116. The tension cone consists of a 60-degree conical tip that is 10 square cm in projected horizontal cross-sectional area, and a cylindrical friction sleeve with a surface area of 150 square cm. The tip, sleeve, and rods have outer diameters of 3.6 cm. A diagram of the tension cone is shown in Appendix F. This CPT instrument can only be loaded to about 5 tons. The tension cone is capable of very accurate f_s measurements because f_s is monitored with a separate set of strain gauges, unlike the subtraction cone.

Quality Assurance

117. The potential sources of error in CPT data that received particular attention were probe calibration, zero drift, and depth referencing. The nature of these errors, how they were dealt with, and observations in the field are described in more detail below.

Probe calibration

118. All CPT probes were calibrated in the laboratory under carefully-controlled conditions. In the field, the probes were calibrated once again after the entire system of signal conditioners, amplifiers, and recorders were

engaged, to account for system effects on calibration. This through-the-system calibration was performed upon arrival at the dam site.

Zero drift checking

119. Changes in the zero reading from the beginning of a sounding to the time when the probe is pulled out of the ground can have many causes, such as temperature changes in the probe or the electrical equipment, excessive straining of the metals in the probe, dirt lodging in the gaps at either end of the friction sleeve, bent or misaligned sleeve units, partial or complete unbonding of the strain gauges, and water in the electrical connections. Zero drift problems are particularly important to account for when using the subtraction cone in order to obtain reliable f_s readings, since f_s is determined as the difference between two relatively large numbers with this design.

120. In the field, the zero reading was checked at the beginning and at the end of each sounding. These checks were recorded on the strip chart, the digital cassette deck, and a paper tally chart. A linear change in the zero was assumed to occur over the depth of the sounding to correct the measurements. In general, very little zero drift was observed. The zero drift was never more than 20 percent of the lowest reading during the push. If a positive zero drift occurred, there was never a case which resulted in apparent negative measurements when a linear change was assumed between beginning and ending zeros.

121. Careful calibration and zero drift monitoring are particularly important for reliable f_s readings from the subtraction probe. A statistical comparison of f_s measured from subtraction cone soundings and nearby tension cone soundings showed they were essentially the same, indicating the calibration and zero drift monitoring efforts were worthwhile.

Depth referencing

122. Careful monitoring of probe elevation was necessary to accurately measure the thickness and elevations of individual soil layers for correlation with other soundings to map the areal extent and depth of potential problem zones. The datum for the depth measurements is the floor of the CPT truck. During probing, there is a possibility that the truck, which has been lifted off the ground with jacks, will sink a few inches due to the heavy load on the jack plates. The truck can also bend elastically, while the probe penetrates dense soil during a push of a one-meter rod. ERTEC performed numerous depth checks during each sounding to account for elastic truck deformation and

sinking of the jacking plates into the ground. The depth reference check involved counting the number of rods in the ground, accounting for the length of the probe, and measuring the distance from the truck reference down to the ground surface with a ruler. All depth adjustments were recorded on paper, and were incorporated in the final CPT data files in the CPT data base.

CPT Results and Data Base

123. ERTEC transmitted the CPT data to WES in three forms: (a) computer generated plots of q_c, f_s, and friction ratio ($100\ f_s/q_c$) versus depth for each CPT sounding, (b) cross sections of the CPT strings (locations shown in Figure 41), with q_c and friction ratio plotted versus depth, and arranged at the appropriate elevation on the cross section, and (c) CPT data on magnetic 9 track half inch tape. Two data bases were developed, one that lists the overall CPT program, and another that gives detailed results for each sounding.

124. The CPT program data base is shown in Tables 21 and 22. Table 21 lists the CPT sounding identifier, the ground surface elevation, a code that distinguishes free field soundings (FF), made downstream of the toe of the dam and switchyard, from soundings made in the switchyard areas (SW). Table 21 also lists the number of and identifies nearby SPT or undisturbed borings for correlation purposes. Table 22 is a companion to Table 21 and lists the SPT and undisturbed borings, ground surface elevations, free field or switchyard code, and the number and identifier of nearby CPT soundings.

125. The actual results of the CPT measurements are stored in another series of computer files for the detailed study of stratigraphy and liquefaction potential evaluation to be discussed in subsequent reports (Volumes 4 and 5 of this series). The data files are given in Appendix E for each of the CPT soundings. Table 23 shows the first part of the data file for CPT-1. The data file consists of a header with detailed information about the particular sounding, and several columns with the actual CPT measurements.

126. The header gives the name of the project, the project number as assigned by ERTEC, the sounding name such as CPT-1, the date of the test, the identification number for the instrument (the last three numbers correspond to the cone type identification given in Table 20), and the depth to the water table (i.e., 38 ft for CPT-1). The next 4 items in the header are depth

intervals over which the measurements are averaged or smoothed. CONE SMOOTH, FRIC SMOOTH, PORE SMOOTH, and COND SMOOTH refer to the depth interval for averaging tip resistance (q_c), sleeve friction resistance (f_s), pore pressure, and conductivity, respectively. For the Barkley site, there was no averaging of these values, so the smoothing intervals are zero. This means that the q_c, f_s, pore pressure, and conductivity values listed in columns below the header are point values. The number of data points for each sounding is given. CPT-1 has 974 data points.

127. The next three items in the header, CONE-FRIC LEAD, CONE-PORE LEAD, and CONE-COND LEAD, give the depth offset in ft of the location of the measuring devices for friction, pore pressure, and conductivity, relative to a reference point in the middle of the cone tip. The data in the columns have already been adjusted by these offset depths. The following unit weights are listed in pcf: GAMMA OF WATER is the unit weight of water, GAMMA ABOVE WT is the total unit weight of soil above the water table, and GAMMA BELOW WT is the total unit weight of soil below the water table. These values were simply assumed as typical values. The last three items in the header are more smoothing intervals. RF SMOOTH, RU SMOOTH, and RC SMOOTH are smoothing intervals in units of ft for friction ratio, pore pressure ratio, and conductivity ratio (as defined in Appendix F). For CPT-1, the friction ratio was smoothed over a depth interval of 0.5 ft.

128. There are six columns shown in Table 23: the depth (ft), the cone resistance, q_c (tsf), the friction resistance, f_s (tsf), the pore pressure (tsf), conductivity (mho/cm), and the smoothed friction ratio. Diskettes containing the CPT data base are given in Appendix E. See Appendix F, the ERTEC report, for plots of the individual soundings.

PART XII: SUMMARY DESCRIPTION OF FOUNDATION

129. The foundation beneath Barkley Dam is complex and previous descriptions have therefore been general and have divided it in three general zones or units as described in paragraph 23. As a result of the extensive exploration that was performed for this study, a more detailed description can now be made.

130. Unit 1 of the foundation is a medium stiff clay that is 20 to 30 ft in thickness. In the switchyard area it is somewhat thicker, extending down to Elevation 325 to 320 ft while along the long portion of the embankment it extends down to Elevation 330 to 325 ft. Sand layers are present in this unit but the number and extent are very limited and isolated as indicated by the explorations. The uncorrected SPT blowcounts average about 10 along the embankment, but increase to about 17 in the free field just beyond the switchyard and average about 25 under the switchyard. The average liquid limit, plastic limit and water content in this unit are about 30, 17, and 23 percent, respectively.

131. Unit 2 of the foundation is dominated by a very soft clay, interbedded with silts and sands with the thickness of the silt and sand layers being very thin, generally less than 6 in. Individual layers of sand could not be correlated using the CPT and SPT data, however, the downstream streambank exposure indicated that continuity in the direction parallel to the river is probable and that these layers are very undulating. The boundary between Unit 1 and this Unit can generally be seen by the decrease in SPT and CPT values. The average uncorrected SPT blowcount in the clay of this unit was about 5 to 7 although values of 0 to 2 were also recorded. SPT drives that were in the sand layers averaged uncorrected blowcounts of about 8 in the free field beyond the switchyard, 11 under the switchyard and about 16 along the rest of the dam. The thickness of this zone also varies with the boundary between Unit 2 and Unit 3 marked by dense sands and gravels. Along the main part of the dam, this occurs between Elevation 300 and 295 ft and in switchyard area it occurs a little higher, usually between Elevation 305 and 300 ft. The sands that are present in this unit are very dirty containing large amounts of silt and clay fractions with an average fines content of about 30 percent.

132. Unit 3 of this foundation is made up sands and gravels although layers of clay are also found. One continuous layer of clay was found in the

switchyard and free field area between Elevations 295 and 290 ft. The materials in this unit are denser as can be seen by the SPT and CPT explorations. The average uncorrected blowcount in the sand and gravels was about 35, and in the clay it was about 11. The sands are cleaner than those found in Unit 2, and have an average fines content of about 15 percent.

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Table 1

Design Strength Test Data

Material	Saturated Unit Weight pcf	Moist Unit Weight pcf	Drain Shear Strength (S) from DCD Tests		Consolidated-Undrained Shear Strength (R) from TCU Tests		Unconsolidated-Undrained Shear Strength (Q) from TUU Tests	
			tan ϕ	c (tsf)	tan ϕ	c (tsf)	tan ϕ	c (tsf)
Impervious Fill	128	126	0.500	0	0.300	0	0.065	0.50
Random Fill	128	126	0.250	0	0.150	0.20	0.033	0.25
Zone A - 0-19'	128	126	0.413	0	0.285	1.00	0.221	0.70
Zone B - 19-53'	128	126	0.602	0	0.380	0.38	0.276	0.44
Zone C - 53-90'	128	120	0.710	0.15	0.700	0.10	--	--

TCU - Triaxial Consolidated Undrained.
DCD - Direct Shear.
TUU - Triaxial Unconsolidated Undrained.
Table from DM 3C - Soil Explorations and Right Bank Earth Structures Zones A, B, and C are from test results from holes BDH-9 and 11.

Table 2

Hole S-1 Station 29+11, 1+40A

Material	Average Values: Natural Water Content %	Average Values: Dry Density pcf	Average Values: Atterberg Limits LL	Average Values: Atterberg Limits PL	Drained Shear Strength (S) from DCD Tests: tan ϕ	Drained Shear Strength (S) from DCD Tests: c (tsf)	Consolidated-Undrained Shear Strength (R) from TCU Tests: tan ϕ	Consolidated-Undrained Shear Strength (R) from TCU Tests: c (tsf)
Clay (Clay cap)	23.2	102.4	41.8	21.2	0.508	0.20	0.509	0.40
SM	21.3	101.5	--	--	0.523	0	0.595	0
ML-CL	26.1	102.4	21.0	17.0	0.570	0	0.412	0.36
CL (below clay cap)	24.5	100.8	30.0	17.0	0.532	0.08	--	--

Table 3

Record Sample Data - Right Embankment (Phase 1)

Material	No. Samples	Average Values: Natural Water Content %	Average Values: Dry Unit Weight pcf	Average Values: Moist Unit Weight pcf	Average Values: Saturated Unit Weight pcf	Atterberg Limits: LL	Atterberg Limits: PL	Drained Shear Strength (S) from DCD Tests: tan ϕ	Drained Shear Strength (S) from DCD Tests: c (tsf)
Foundation Clay	6	21.6	102.4	124.5	127.0	38	22	0.476	0.16
Embankment	1	21.7	105.1	127.9	129.0	46	24	0.510	0.23

Table 4

Record Sample Data - Right Embankment (Phase 2) (All Embankment Material)

Average Values: Natural Water Content %	Average Values: Dry Unit Weight pcf	Average Values: Moist Unit Weight pcf	Average Values: Saturated Unit Weight pcf	Atterberg Limits: LL	Atterberg Limits: PL	Drained Shear Strength (S) from DCD Tests: tan ϕ	Drained Shear Strength (S) from DCD Tests: c (tsf)	Consolidated-Undrained Shear Strength (R) from TCU Tests: tan ϕ	Consolidated-Undrained Shear Strength (R) from TCU Tests: c (tsf)
17.2	110.7	129.7	131.9	35.5	19.4	0.509	0	0.496	0.61

TCU - Triaxial Consolidated Undrained.
DCD - Direct Shear.

Table 5

Summary of Geophysical Tests Performed at Barkley Dam

Study Area	Location	Date	Test Type	Wave Type	Line ID	Line Length ft	Remarks
			Surface Techniques				
Location 1	STA 64+006	Dec 1977	Surface	P-Wave	RS-1	625	Downstream, parallel to toe
			Refraction	P-Wave	RS-2	625	Downstream, perpendicular to toe
				P-Wave	RS-3	625	Downstream, parallel to toe
				S-Wave	RS-3	625	Downstream, parallel to toe
				P-Wave	RS-4	165	Along dam crest
				S-Wave	RS-4	165	Along dam crest
			Surface vibratory	Rayleigh wave	RY-1	22	Downstream toe, near crosshole set

Study Area	Date	Test Type	Borings	Station L	Offset B	TOH Elevation	Depth ft	Remarks
				Subsurface Techniques				
Location 1	Dec 77	Crosshole (3-hole set)	BEQ 1U	64+20	2+31	349.6	127.2	P-wave and S-wave at downstream toe
			BEQ 2U	64+00	2+51	350.2	121.9	
			BEQ 6	64+20	2+51	350.3	132.2	
		Downhole	BEQ 2U	64+00	2+51	350.2	121.9	S-wave successful P-wave unsuccessful
Location 2	Apr 84	Crosshole (2-hole set)	WES 1-1	36+00	0+38	387.1	127.3	P-wave and S-wave on downstream slope
			WES 1-2	36+11	0+39	387.6	126.6	
Location 3	Apr 84	Crosshole (2-hole set)	WES 2-1	34+45	4+95	341.3	88.3	P-wave and S-wave downstream of switchyard near tailrace slope
			WES 2-2	34+45	4+85	341.3	88.0	
Location 4	May 85	Downhole	CPT 12	38+70	2+07	365.7	86.6	S-wave successful P-wave unsuccessful
Location 5	May 85	Downhole	CPT 26	34+56	4+98	341.5	67.3	S-wave successful P-wave unsuccessful

Table 6

Rayleigh Wave Velocities Estimated From Field Measurements

Frequency Hz	Velocity fps	Depth ft
30	540	9.0
50	540	5.5
70	545	4.0
90	420	2.5
120	415	2.0
150	395	1.5

Table 7

S-Wave Velocity Zones from Downhole Tests

Depth (ft)	V_s (fps)
0-5	385
5-25	780
25-50	568
50-80	774
80-115+	1170

Table 8A

Piezometers Prior to 1977

PZ Number	Date Installed	Location		Midtip Elevation	Unit
		L	B		
BP-1	10 SEP 1970	33+97	0+42	269.3	EMBANKMENT
BP-2	15 SEP 1970	33+82	1+30	269.7	EMBANKMENT
BP-3	14 AUG 1970	34+82	3+85	279.3	3
BP-4	29 JUL 1970	36+80	0+29 (U/S)	314.4	2
BP-4A	29 JUL 1970	36+80	0+29 (U/S)	284.4	3
BP-5	9 SEP 1970	36+80	0+26	301.4	2
BP-5A	9 SEP 1970	36+80	0+26	257.3	3
BP-6	26 AUG 1970	36+80	4+37	281.1	3
BP-7	24 JUL 1970	40+06	1+70	274.8	3
BP-8	21 JUL 1970	42+64	2+00	301.4	2
BP-9	19 AUG 1970	46+28	1+94	342.8	1
BP-10	3 AUG 1970	49+74	0+59	318.9	2
BP-11	28 AUG 1970	49+74	1+00	322.2	2
BP-12	24 AUG 1970	49+74	1+60	321.9	2
BP-13	18 AUG 1970	52+06	2+00	327.9	2
BP-14	6 AUG 1970	57+00	0+59	288.7	3
BP-15	17 AUG 1970	57+00	3+00	334.8	1
BP-16	12 AUG 1970	57+00	1+60	323.5	2
BP-17	11 AUG 1970	57+00	1+00	320.6	2
BP-18	16 JUL 1970	59+00	1+95	280.4	3
BP-19	21 AUG 1970	65+00	2+00	327.8	2
BP-20	20 AUG 1970	75+80	2+45	325.6	2
BP-21	18 AUG 1970	49+74	3+00	327.3	2
BP-22	NOV 1977	34+41	0+41	275.6	EMBANKMENT
BP-23	NOV 1977	34+79	2+60	274.8	3
BP-24	NOV 1977	35+50	0+49	274.5	3
BP-25	NOV 1977	34+50	1+25	274.7	EMBANKMENT

Table 8B

Piezometers Installed in SPT Holes

PZ Number	Date Installed	Location L	Location B	Midtip Elevation	Unit
BEQ-7	8 NOV 1982	34+33	4+86	283.7	3
BEQ-8	10 NOV 1982	34+40	4+81	284.3	3
BEQ-9	17 NOV 1982	49+25	2+15	277.5	3
BEQ-10	23 NOV 1982	54+00	2+10	288.4	3
BEQ-11	1 DEC 1982	59+00	2+30	288.2	3
BEQ-12	9 DEC 1982	69+00	2+40	288.2	3
BEQ-13	20 JAN 1983	74+06	2+60	285.7	3
BEQ-14	16 JAN 1983	79+05	3+10	283.2	3
BEQ-15	10 MAY 1984	35+60	1+52	309.1	2
BEQ-16	21 MAY 1984	35+60	1+47	330.8	1
BEQ-17	17 APR 1984	36+95	1+52	308.1	2
BEQ-18	30 APR 1984	36+95	1+47	326.0	1
BEQ-19	31 MAY 1984	39+85	1+75	306.9	2
BEQ-20	12 JUN 1984	39+85	1+70	330.3	1
BEQ-21	10 JUL 1984	34+35	4+91	310.1	2
BEQ-22	25 JUL 1984	34+35	4+96	323.4	1
BEQ-23	15 MAR 1984	36+95	5+00	311.6	2
BEQ-24	9 APR 1984	37+00	5+00	338.7	1
BEQ-25	20 JUN 1984	39+50	4+80	311.7	2
BEQ-26	3 JUL 1984	39+50	4+75	336.8	1
BEQ-27	31 JUL 1984	34+35	7+00	309.7	2
BEQ-28	16 AUG 1984	34+35	7+05	326.7	1
BEQ-29	12 AUG 1984	36+95	7+00	309.2	2
BEQ-30	22 AUG 1984	36+94	7+05	329.1	1
BEQ-31	28 AUG 1984	39+80	6+90	311.9	2
BEQ-32	5 SEP 1984	39+80	6+85	334.5	1
BEQ-33	20 SEP 1984	39+84	2+76	309.6	2
BEQ-34	27 SEP 1984	39+84	2+64	284.7	3

Table 8C

Piezometers After 1977

PZ Number	Date Installed	Location L	Location B	Midtip Elevation	Unit
EQ-2A	5 JUL 1979	49+11	0+03	313.8	EMBANKMENT
EQ-6	17 AUG 1979	33+77	0+30	262.3	EMBANKMENT
EQ-7	30 JUL 1979	33+77	1+40	254.1	EMBANKMENT
EQ-12	2 AUG 1979	33+57	1+31	323.5	EMBANKMENT
EQ-13	6 AUG 1979	33+61	1+57	282.5	EMBANKMENT
EQ-14	10 AUG 1979	33+69	1+49	259.0	EMBANKMENT
BD-1	4 SEP 1981	65+10	2+20	340.7	1
BD-2	9 SEP 1981	65+00	1+90	342.0	1
BD-3	15 SEP 1981	64+00	2+00	314.1	2
BD-4	22 SEP 1981	66+00	2+00	313.1	2
BD-5	14 SEP 1981	65+00	2+60	340.9	1
BD-6	15 SEP 1981	63+00	2+00	337.8	1
BD-7	22 SEP 1981	67+00	2+00	314.2	2
BD-8	28 SEP 1981	65+50	2+30	318.6	2
BD-9	5 OCT 1981	64+50	0+60	338.5	1
WES-1	JUN 1979	64+04	2+10	326.3	1
WES-2	JUN 1979	64+09	2+10	306.2	2
WES-3	JUN 1979	64+14	2+11	286.2	3
WES-4	JUN 1979	64+05	2+41	330.1	1
WES-5	JUN 1979	64+10	2+41	310.2	2
WES-6	JUN 1979	64+14	2+41	290.1	3

Table 8D

Unit 1 Piezometers

Piezometer Number	6 March Readings	2 July Readings	Midtip Elevation
	Located at Toe of Embankment		
BP-9	350.4	349.6	342.8
BP-15	347.3	346.8	334.8
BD-1	347.8	347.3	340.7
BD-2	343.0	342.9	341.7
BD-5	347.6	346.6	340.9
BD-6	346.8	348.9	337.8
BD-9	347.3	350.3	338.5
WES-1	346.1	348.0	326.3
WES-4	345.9	347.5	330.1
	Located in Switchyard and Toe of Switchyard		
BEQ-16	332.9	333.2	330.8
BEQ-18	333.6	334.0	326.0
BEQ-20	339.9	340.2	330.3
BEQ-22	328.5	322.6	323.4 (DRY)
BEQ-24	345.0	344.7	338.7
BEQ-26	339.9	343.3	336.8
BEQ-28	331.7	325.8	326.7 (DRY)
BEQ-30	328.2	328.2	329.1 (DRY)
BEQ-32	334.1	334.8	334.5

6 March 1985

Headwater = 355.6
Tailwater = 330.8

2 July 1985

Headwater = 359.2
Tailwater = 302.5

Table 8E

Unit 2 Piezometers

Piezometer Number	6 March Readings	2 July Readings	Midtip Elevation
BEQ-21	331.3	312.4	310.1
BEQ-23	328.7	315.2	311.6
BEQ-25	336.3	332.9	311.7
BP-8	344.2	339.7	301.4
BD-4	342.0	342.7	313.1
BD-8	342.9	342.7	318.6
BD-3	342.1	342.4	314.1
WES-2	341.4	340.6	306.2
WES-5	341.9	342.0	310.2

6 March 1985

Headwater = 355.6
Tailwater = 330.8

2 July 1985

Headwater = 359.2
Tailwater = 302.5

Table 8F

Unit 2 Piezometers (Switchyard)

Piezometer Number	6 March Readings	2 July Readings	Midtip Elevation
BEQ-27	330.6	312.5	309.7
BEQ-29	329.5	315.2	309.2
BEQ-31	335.5	332.6	311.9
BEQ-21	331.3	312.4	310.1
BEQ-23	328.7	315.2	311.6
BEQ-25	336.3	332.9	311.7
BEQ-15	330.5	314.6	309.1
BEQ-17	335.6	332.0	308.1
BEQ-19	341.8	336.7	306.9

6 March 1985

Headwater = 355.6
Tailwater = 330.8

2 July 1985

Headwater = 359.2
Tailwater = 302.5

Table 8G

Unit 3 Piezometers

Piezometer Number	6 March Readings	2 July Readings	Midtip Elevation
BEQ-7	332.9	312.3	283.7
BEQ-8	335.4	320.3	284.3
BEQ-34	336.7	323.0	284.7
BP-7	336.7	322.8	274.8
BEQ-9	339.7	329.0	277.5
BEQ-10	341.2	332.2	288.4
BP-18	341.1	331.3	280.4
BEQ-11	342.0	335.1	288.2
WES-3	340.6	338.4	286.2
WES-6	340.7	338.3	290.1
BEQ-12	341.5	339.4	288.2
BEQ-13	341.9	339.9	285.7
BEQ-14	341.9	339.9	283.2

6 March 1985

Headwater = 355.6
Tailwater = 330.8

2 July 1985

Headwater = 359.2
Tailwater = 302.5

Table 8H

WES Piezometers

Piezometer Number	6 March Readings	2 July Readings	Midtip Elevation
WES-1	346.1	348.0	326.3
WES-2	341.4	340.6	306.2
WES-3	340.6	338.4	286.2
WES-4	345.9	347.5	330.1
WES-5	341.9	342.0	310.2
WES-6	340.7	338.3	290.1

6 March 1985

Headwater = 355.6
Tailwater = 330.8

2 July 1985

Headwater = 359.2
Tailwater = 302.5

Table 9

SPT Borings

SPT No.	Date Drilled	Location L	Location B	EL. TOH*	Depth	No. Samp	Method of Drilling	Drilling Agency
BEQ-1	5 OCT 1977	44+50	2+10	345.6	124.0	40	STANDARD	WES
BEQ-2	10 OCT 1977	54+00	2+10	347.2	119.0	40	STANDARD	WES
BEQ-3	12 OCT 1977	64+00	2+00	349.6	120.0	39	STANDARD	WES
BEQ-4	18 OCT 1977	74+00	2+60	351.7	115.7	38	STANDARD	WES
BEQ-5	20 OCT 1977	84+00	4+80	343.6	61.5	12	STANDARD	WES
BEQ-6	18 NOV 1977	64+20	2+51	350.3	132.2	36	STANDARD	WES
DS-3	9 JUN 1979	34+28	4+81	340.3	94.1		**	WES
BEQ-7	7 NOV 1982	34+33	4+86	341.5	60.0	53	CONTINUOUS	NASHVILLE
BEQ-8	10 NOV 1982	39+40	4+81	349.8	66.5	33	STANDARD	NASHVILLE
BEQ-9	17 NOV 1982	49+25	2+15	350.5	74.0	42	STANDARD	NASHVILLE
BEQ-10	23 NOV 1982	54+00	2+10	347.2	60.0	79	CONTINUOUS	NASHVILLE
BEQ-11	1 DEC 1982	59+00	2+30	347.0	61.5	62	STANDARD	NASHVILLE
BEQ-12	9 DEC 1982	69+00	2+40	348.5	61.5	61	STANDARD	NASHVILLE
BEQ-13	20 JAN 1983	74+06	2+60	344.0	60.0	90	CONTINUOUS	NASHVILLE
BEQ-14	13 JAN 1983	79+05	3+10	345.0	64.0	57	STANDARD	NASHVILLE
BEQ-15	9 MAY 1984	35+60	1+52	365.7	86.5	83	STANDARD	NASHVILLE
BEQ-16	22 MAY 1984	35+60	1+48	365.7	84.0	126	CONTINUOUS	NASHVILLE
BEQ-17	17 APR 1984	36+95	1+52	366.1	86.5	84	STANDARD	NASHVILLE
BEQ-18	26 APR 1984	36+95	1+47	366.1	84.5	89	STANDARD	NASHVILLE
BEQ-19	31 MAY 1984	39+85	1+75	364.4	81.5	77	STANDARD	NASHVILLE
BEQ-20	13 JUN 1984	39+85	1+70	364.7	81.0	131	CONTINUOUS	NASHVILLE
BEQ-21	12 JUL 1984	34+35	4+96	341.5	61.5	55	STANDARD	NASHVILLE
BEQ-22	22 JUL 1984	34+35	4+91	341.5	58.5	103	CONTINUOUS	NASHVILLE
BEQ-23	12 MAR 1984	36+95	5+00	347.3	66.5	59	STANDARD	NASHVILLE
BEQ-24	9 APR 1984	37+00	5+00	347.3	67.0	23	STANDARD	NASHVILLE
BEQ-25	20 JUN 1984	39+50	4+86	349.8	69.0	62	STANDARD	NASHVILLE
BEQ-26	2 JUL 1984	39+50	4+75	349.8	67.5	110	CONTINUOUS	NASHVILLE

(Continued)

* Top of hole.

** Boring was alternating SPT-Undisturbed.

Table 9 (Concluded)

SPT No.	Date Drilled	Location L	Location B	EL. TOH	Depth	No. Samp	Method of Drilling	Drilling Agency
BEQ-27	30 JUL 1984	34+35	7+00	342.7	59.0	56	STANDARD	NASHVILLE
BEQ-28	7 AUG 1984	34+35	7+05	342.7	61.5	101	CONTINUOUS	NASHVILLE
BEQ-29	14 AUG 1984	36+95	7+00	347.2	64.0	65	STANDARD	NASHVILLE
BEQ-30	22 AUG 1984	36+94	7+05	347.7	67.5	103	CONTINUOUS	NASHVILLE
BEQ-31	28 AUG 1984	39+80	6+90	350.0	69.0	67	STANDARD	NASHVILLE
BEQ-32	5 SEP 1984	36+85	6+85	350.0	65.0	106	CONTINUOUS	NASHVILLE
BEQ-33	20 SEP 1984	39+84	2+76	362.9	78.0	123	CONTINUOUS	NASHVILLE
BEQ-34	24 SEP 1984	39+84	2+64	362.7	79.0	76	STANDARD	NASHVILLE
BD-1	2 SEP 1981	65+10	2+20	348.5	39.0	25	CONTINUOUS	NASHVILLE
BD-2	8 SEP 1981	65+10	2+00	349.7	40.0	23	CONTINUOUS	NASHVILLE
BD-3	15 SEP 1981	64+60	2+35	349.8	39.0	26	CONTINUOUS	NASHVILLE
BD-4	17 SEP 1981	66+00	2+27	348.5	39.0	26	CONTINUOUS	NASHVILLE
BD-5	9 SEP 1981	65+10	2+71	349.4	39.0	26	CONTINUOUS	NASHVILLE
BD-6	14 SEP 1981	63+00	2+30	348.6	39.0	25	CONTINUOUS	NASHVILLE
BD-7	23 SEP 1981	66+00	2+35	348.0	39.0	26	CONTINUOUS	NASHVILLE
BD-8	28 SEP 1981	65+50	2+30	348.6	31.6	21	CONTINUOUS	NASHVILLE
BD-9	5 OCT 1981	64+50	0+60	374.0	64.5	42	CONTINUOUS	NASHVILLE

Table 10

Barkley Dam - Dynamic Analysis Boring Data Base

Boring Name	P	Z	Ground Elevation	Location North	Location East	Drilling Method	Depth Maximum	Water Table Depth
B-D-1	1	1	348.50	6510	220	LONGYEAR 0054	39.00	2.50
B-D-2	1	1	349.70	6510	200	LONGYEAR 0054	40.00	2.70
B-D-3	1	1	349.80	6460	235	LONGYEAR	39.00	9.80
B-D-4	1	1	348.50	6600	227	LONGYEAR 0054	39.00	8.00
B-D-5	1	1	349.40	6510	271	LONGYEAR 0054	39.00	4.50
B-D-6	1	1	348.60	6300	230	LONGYEAR	39.00	2.50
B-D-7	1	1	348.00	6700	235	LONGYEAR 0054	39.00	8.00
B-D-8	1	1	348.60	6550	230	LONGYEAR	31.60	9.00
B-D-9	1	1	374.00	6450	60	LONGYEAR	64.50	27.00
BEQ-01	1	1	354.57	4450	210	CE 4522	124.00	12.50
BEQ-02	1	1	346.22	5400	210	CE 4522	119.00	2.20
BEQ-03	1	1	349.63	6400	210	CE 4522	120.00	4.60
BEQ-04	1	1	351.66	7400	260	CE 4522	115.00	6.60
BEQ-05	1	1	343.55	8400	480	CE 4522	61.50	1.50
BEQ-06	1	1	350.26	6420	251	CE 4522	132.20	5.30
BEQ-07	1	1	341.50	3433	486	FAILING 1500	60.00	26.50
BEQ-08	1	1	349.80	3940	481	FAILING 1500	66.50	15.00
BEQ-09	1	1	350.50	4925	215	FAILING 1500	74.00	14.50
BEQ-10	1	1	347.20	5400	210	FAILING 1500	60.00	12.20
BEQ-11	1	1	347.00	5900	230	FAILING 1500	61.50	12.00
BEQ-12	1	1	348.50	6900	240	FAILING 1500	61.50	8.50
BEQ-13	1	1	344.00	7406	260	FAILING 1500	60.00	4.00
BEQ-14	1	1	345.04	7905	310	FAILING 1500	64.00	5.00
BEQ-15	1	1	365.70	3560	152	MOBILE B-53	86.50	25.70
BEQ-16	1	1	365.70	3560	148	MOBILE B-53	84.00	15.70
BEQ-17	1	1	366.10	3695	152	MOBILE B-53	86.50	26.10
BEQ-18	1	1	366.10	3695	147	MOBILE B-53	84.50	26.10
BEQ-19	1	1	364.40	3985	175	MOBILE B-53	81.50	22.20
BEQ-20	1	1	364.70	3985	170	MOBILE B-53	81.00	24.70

(Continued)

Table 10 (Concluded)

Boring Name	P	Z	Ground Elevation	Location North	Location East	Drilling Method	Depth Maximum	Water Table Depth
BEQ-21	1	1	341.50	3435	496	MOBILE B-53	61.50	21.50
BEQ-22	1	1	341.50	3435	491	MOBILE B-53	58.50	21.50
BEQ-23	1	1	347.33	3695	500	MOBILE B-53	66.50	17.33
BEQ-24	1	1	347.30	3700	500	MOBILE B-53	67.00	17.30
BEQ-25	1	1	349.75	3950	486	MOBILE B-53	69.00	9.80
BEQ-26	1	1	349.80	3950	475	MOBILE B-53	67.50	9.80
BEQ-27	1	1	342.70	3435	700	MOBILE B-54	59.00	22.70
BEQ-28	1	1	342.70	3435	705	MOBILE B-53	61.50	22.70
BEQ-29	1	1	347.20	3695	700	MOBILE B-53	64.00	27.20
BEQ-30	1	1	347.70	3694	705	MOBILE B-53	67.50	27.20
BEQ-31	1	1	350.00	3980	690	MOBILE B-53	69.00	15.00
BEQ-32	1	1	350.00	3685	685	MOBILE B-53	65.00	15.00
BEQ-33	1	1	362.90	3984	276	MOBILE B-53	78.00	22.90
BEQ-34	1	1	362.70	3984	264	MOBILE B-53	79.00	22.70
DS-3	1	1	340.00	3428	481	CE 8076	86.30	20.00

Table 11

Barkley Dam SPT Sampler Listing SPT Sampler Data

Boring	Sampler Top	Sampler Bottom	0-6	6-12	12-18	Number of Samples	Sample 1	2	3	4	5	6
				Boring Group BEG-20								
BEQ-20	0.00	1.50	2	5	6	2	001	002				
BEQ-20	1.50	3.00	3	4	5	2	003	004				
BEQ-20	3.00	4.50	3	4	5	2	005	006				
BEQ-20	4.50	6.00	3	7	14	2	007	008				
BEQ-20	6.00	7.50	4	8	8	2	009	010				
BEQ-20	7.50	9.00	3	7	9	2	011	012				
BEQ-20	9.00	10.50	3	6	7	2	013	014				
BEQ-20	10.50	12.00	4	7	9	2	015	016				
BEQ-20	12.00	13.50	5	9	10	3	017	018	019			
BEQ-20	13.50	15.00	4	4	7	2	020	021				
BEQ-20	15.00	16.50	7	9	8	2	022	023				
BEQ-20	16.50	18.00	8	10	14	2	024	025				
BEQ-20	18.00	19.50	5	6	10	2	026	027				
BEQ-20	19.50	21.00	6	10	14	2	028	029				
BEQ-20	21.00	22.50	8	8	10	2	030	031				
BEQ-20	22.50	24.00	5	7	12	2	032	033				
BEQ-20	24.00	25.50	5	21	17	2	034	035				
BEQ-20	25.50	27.00	17	11	6	2	036	037				
BEQ-20	27.00	28.50	4	7	13	2	038	039				
BEQ-20	28.50	30.00	4	10	10	2	040	041				
BEQ-20	30.00	31.50	6	10	14	3	042	043	044			
BEQ-20	31.50	33.00	5	7	13	3	045	046	047			
BEQ-20	33.00	34.50	5	10	11	3	048	049	050			
BEQ-20	34.50	36.00	5	10	16	3	051	052	053			
BEQ-20	36.00	37.50	5	14	16	3	054	055	056			
BEQ-20	37.50	39.00	4	8	12	2	057	058				
BEQ-20	39.00	40.50	6	10	13	2	059	060				
BEQ-20	40.50	42.00	4	8	13	3	061	062	063			
BEQ-20	42.00	43.50	5	7	11	3	064	065	066			
BEQ-20	43.50	45.00	4	9	13	3	067	068	069			
BEQ-20	45.00	46.50	4	9	12	2	070	071				
BEQ-20	46.50	48.00	3	6	6	2	072	073				
BEQ-20	48.00	49.50	3	6	6	2	074	075				
BEQ-20	49.50	51.00	2	3	5	3	076	077	078			
BEQ-20	51.00	52.50	2	4	5	4	079	080	081A	081B		

(Continued)

Table 11 (Concluded)

Boring	Sampler Top	Sampler Bottom	0-6	6-12	12-18	Number of Samples	Sample 1	2	3	4	5	6
BEQ-20	52.50	54.00	1	3	2	3	082	083	084			
BEQ-20	54.00	55.50	2	3	5	3	085	086	087			
BEQ-20	55.50	57.00	3	4	5	3	088	089	090			
BEQ-20	57.00	58.50	11	14	16	2	091	092				
BEQ-20	58.50	60.00	7	13	10	3	093A	093B	094			
BEQ-20	60.00	61.50	5	11	13	2	095	096				
BEQ-20	61.50	63.00	6	3	6	5	097A	097B	098A	098B	099	
BEQ-20	63.00	64.50	5	10	9	2	100	101				
BEQ-20	64.50	66.00	6	14	16	2	102	103				
BEQ-20	66.00	67.50	5	14	14	3	104A	104B	105			
BEQ-20	67.50	69.00	7	10	15	2	106	107				
BEQ-20	69.00	70.50	5	21	19	2	108	109				
BEQ-20	70.50	72.00	14	18	12	2	110	111				
BEQ-20	72.00	73.50	4	5	6	3	112	113	114			
BEQ-20	73.50	75.00	4	5	8	4	115	116	117A	117B		
BEQ-20	75.00	76.50	4	8	12	3	118	119A	119B			
BEQ-20	76.50	78.00	8	16	21	1	120					
BEQ-20	78.00	79.50	19	24	18	2	121	122				
BEQ-20	79.50	81.00	11	20	29	2	123	124				

Table 12

Barkley Dam - Dynamic Analysis SPT Laboratory Test Results

Boring Number	Sampler Number	Top Sample	Bottom Sample	Natural Water Wn	Liquid Limit LL	Plastic Limit PL	D-60	D-50	D-30	D-10	Percent Pass No. 200	Percent Pass No. .005	Word Classification Minor	Word Classification Major	USCS Soil Class	Color Minor	Color Major
									Boring Group BEQ-20								
BEQ-20	061	40.50	40.80	21.00	0	0	0.014	0.010	0.003	-1.0	87.7	33.0		Clay	CL	Reddish	Brown
BEQ-20	062	40.80	41.30	20.60	0	0	0.012	0.009	0.003	-1.0	92.1	32.0		Clay	CL	Reddish	Brown
BEQ-20	063	41.30	41.80	20.80	0	0	0.011	0.009	0.004	-1.0	84.6	32.0		Clay	CL		Brown
BEQ-20	066	42.60	43.10	21.30	0	0	0.160	0.091	0.003	-1.0	94.9	36.0		Clay	CL		Brown
BEQ-20	068	43.50	43.80	22.00	0	0	0.013	0.009	0.002	-1.0	93.1	44.0		Clay	CL		Brown
BEQ-20	068	43.80	44.30	21.00	0	0	0.011	0.009	0.003	-1.0	95.1	38.0		Clay	CL		Brown
BEQ-20	080	51.50	51.95	23.60	0	0	0.013	0.010	0.004	-1.0	94.3	34.0		Clay	CL	Dark	Gray
BEQ-20	081B	52.05	52.20	20.70	0	0	0.011	0.084	0.021	0.001	45.0	15.5	Silty	Sand	SM	Dark gray and	Red-brown
BEQ-20	081A	52.20	52.30	24.90	0	0	0.050	0.025	0.009	0.001	64.8	23.0	Sandy	Clay	CL	Dark	Gray
BEQ-20	089	56.00	56.40	30.40	44	21	0.018	0.013	0.006	0.002	2.1	23.5		Clay	CL	Dark	Gray
BEQ-20	081	57.00	57.50	25.90	0	0	0.190	0.180	0.140	-1.0	8.6	-1.0		Sand	SP-SM	Gray and yellowish	Brown
BEQ-20	092	57.50	58.00	23.90	0	0	0.195	0.190	0.160	0.120	7.1	-1.0		Sand	SP-SM	Yellowish	Brown
BEQ-20	093A	58.50	58.00	22.10	0	0	0.210	0.200	0.170	0.135	7.6	-1.0	Gravelly	Sand	SP-SM	Yellowish	Brown
BEQ-20	093B	58.70	58.90	28.20	0	0	0.100	0.038	0.015	0.005	56.3	10.0	Sandy	Clay	CL	Dark gray and	Red-brown
BEQ-20	094	58.90	59.20	18.90	0	0	7.00	5.000	0.250	-1.0	12.7	-1.0	Clayey sandy	Gravel	GC	Gray and yellowish	Brown
BEQ-20	096	60.50	61.00	24.30	0	0	0.185	0.170	0.110	-1.0	14.8	-1.0	Silty	Sand	SM		Gray
BEQ-20	097A	61.50	61.60	26.20	0	0	0.040	0.025	0.008	-1.0	70.1	23.0	Sandy	Clay	CL	Dark	Gray
BEQ-20	097B	61.60	61.90	27.90	0	0	0.135	0.110	0.079	-1.0	29.3	-1.0	Silty	Sand	SM	Dark gray and	Red-brown
BEQ-20	098A	61.90	62.30	27.30	31	18	0.054	0.025	0.009	-1.0	673.5	21.5	Sandy	Clay	CL	Dark gray and	Red-brown
BEQ-20	098B	62.30	62.35	24.30	0	0	0.185	0.140	0.088	-1.0	27.1	-1.0	Silty	Sand	SM	Dark	Gray
BEQ-20	099	62.35	62.80	25.50	33	17	0.040	0.024	0.008	-1.0	76.5	23.0	Sandy	Clay	CL	Dark gray and	Red-brown
BEQ-20	101	63.40	63.90	27.50	0	0	0.145	0.140	0.127	0.08	13.4	-1.0	Silty	Sand	SM	Dark gray and	Red-brown
BEQ-20	103	64.85	675.40	24.90	0	0	0.190	0.188	0.160	-1.0	11.4	-1.0		Sand	SP-SM		Gray
BEQ-20	014A	66.00	66.20	27.20	0	0	0.024	0.014	0.005	-1.0	72.5	30.0	Sandy	Clay	CL		Gray
BEQ-20	104B	66.20	66.30	21.90	0	0	0.185	0.180	0.140	-1.0	15.8	-1.0	Silty	Sand	SM	Reddish	Brown
BEQ-20	105	66.40	66.80	24.00	0	0	0.200	0.195	0.188	0.100	9.2	-1.0		Sand	SP-SM	Reddish	Brown
BEQ-20	106	67.70	68.30	26.00	0	0	0.180	0.170	0.115	-1.0	13.9	-1.0	Silty	Sand	SM	Dark	Gray
BEQ-20	109	69.30	69.70	24.40	0	0	0.250	0.235	0.190	0.110	8.0	-1.0		Sand	SP-SM		Gray
BEQ-20	111	61.10	71.30	23.670	33	19	0.030	0.021	0.008	0.001	81.0	22.0	Sandy	Clay	CL	Dark gray and	Red-brown
BEQ-20	116	74.00	74.40	27.10	0	0	0.024	0.160	0.004	-1.0	70.4	30.0	Sandy	Clay	CL	Dark gray and	Red-brown
BEQ-20	117A	74.40	74.45	19.20	0	0	0.250	0.210	0.105	-1.0	27.7	-1.0	Silty	Sand	SM	Reddish brown and	Dark gray
BEQ-20	117B	74.63	74.90	28.70	40	17	0.029	0.019	0.007	0.001	73.7	25.5	Sandy	Clay	CL	Dark gray and	Red-brown
BEQ-20	119A	75.35	75.55	23.70	0	0	0.210	0.080	0.013	0.003	49.9	14.0	Clayey	Sand	SC	Dark gray	
BEQ-20	119B	65.55	75.75	21.00	0	0	0.360	0.310	0.240	-1.0	13.9	-1.0	Silty gravelly	Sand	SM	Dark gray and	Red-brown
BEQ-20	120	76.50	77.10	18.60	0	0	5.300	3.000	0.400	0.150	5.2	-1.0	Gravelly	Sand	SP-SM	Dark	Gray
BEQ-20	121	78.00	78.40	20.90	0	0	0.600	0.400	0.240	0.180	6.5	-1.0	Gravelly	Sand	SP-SM	Reddish	Brown
BEQ-20	122	79.50	80.20	24.40	0	0	0.300	0.290	0.250	0.150	6.6	-1.0		Sand	SP-SM	Dark gray and Yellow	Brown

Table 13

Undisturbed Borings

Boring No.	Date Drilled	Location L	Location B	EL. TOH	Depth Soil (to rock)	No. of Samples
BEQ-1U	3 NOV 1977	64+20	2+31	349.6	114.2 (127.2)	39 Soil 2 Rock
BEQ-2U	NOV 1977	64+00	2+51	350.2	114.2 (121.9)	37 Soil 1 Rock
DS-1	23 MAY 1979	63+80	2+31	349.2	114.3 (122.0)	39 Soil 1 Rock
DS-2	2 JUN 1979	63+60	2+31	350.5	118.3 (124.4)	39 Soil 1 Rock
DS-3	9 JUN 1979	34+30	4+81	340.3	86.3 (94.1)	16 SPT 16 Soil
BEQ-3U	5 DEC 1984	37+00	0+44	385.2	47.6	5 Soil
BEQ-4U	31 OCT 1984	37+20	1+50	366.2	79.3	19 Soil
BEQ-5U	8 NOV 1984	34+61	4+86	341.7	54.7	15 Soil
BEQ-6U	14 NOV 1984	34+74	4+96	341.9	41.1	13 Soil
BEQ-7U	29 NOV 1984	37+00	5+20	347.7	78.7	21 Soil
BEQ-8U	16 NOV 1984	34+43	4+70	341.6	38.1	6 Soil

Table 14

Composite Material Characteristics

Test Soils	Classification	Maximum Density pcf	Minimum Density pcf	Range of Triaxial Specimen Dry Density: As Trimmed pcf	Range of Triaxial Specimen Dry Density: After Consolidation pcf	Percent Fines %	Plasticity Index %	Specific Gravity
Sands	Sand (SP-SM)	116	88.2	91.5-98.2	92.6-99.7	2-9	NP	2.67
Nonplastic silty sands	Silty sand (SM)	121	86.4	91.7-106.7	92.4-106.7	16-33	NP	2.67
Plastic soils	Sandy clay (CL)	129	N/A	94.8-105.8	98.4-108.2	43-89	7-13	2.70

Table 15

$\bar{R}$ Triaxial Compression Test Results (2.8-in.-diam Liquefaction Apparatus, Stress-Controlled)

Test Group	Test No.	Boring	Sample No.	Depth ft	Particle Size Distribution D_{50} mm	Percent Fines -74µm	Percent -5µm	Plasticity Index	Initial Dry Density pcf	Final Dry Density pcf	Effective Consolidated Stress psi	Type of Failure	Initial Maximum Deviator Stress $\sigma_d max$ psi	Axial Strain at $\sigma_d max$ percent	Elapsed Time to $\sigma_d max$ min	Maximum Ratio $\tau_{45°}/\sigma_{45°}$	Effective Confining of Internal Friction deg	Effective Confining Stress at Peak Pore Pressure psi	Void Ratio After Consolidation e_c	Saturation "B"	Specific Gravity G_s	τ_s Steady-State Strength psi
												Sands										
Sands	2-51.7	DS-2	18	51.7		9		NP	96.5	96.8	13.8	S	153.8*	4.4	27	0.612	38	8.3	0.723	97.9	2.67	
	2-51.7R	DS-2	18	51.7	Remolded specimen			NP	88.3	90.2	13.8	LL	9.9	0.4	3	0.594	36	3.6	0.847	97.0	2.67	5.0
	3-63.4	DS-3	22	63.4		1		NP	100.1	100.5	16.9	S	148.6*	2.3	21	0.602	37	12.3	0.659	97.0	2.67	
	3-63.4R	DS-3	22	63.4	Remolded specimen			NP	90.7	91.9	16.9	LL	11.2	0.6	3	0.547	33	4.9	0.814	98.6	2.67	5.0
	2-67.6	DS-2	23	67.7		1		NP	97.6	97.9	18.0	S	147.3*	2.2	19	0.609	38	12.4	0.702	96.5	2.67	
	2-67.6R	DS-2	23	67.6	Remolded specimen			NP	88.0	88.3	18.0	LL	11.3	2.2	3	0.565	34	4.2	0.887	96.2	2.67	5.0
	2-67.6R	DS-2	23	67.6	Remolded specimen			NP	79.3	80.0	18.0	L	6.0	0.3	5	0.583	36	0.0	1.084	99.0	2.67	0.5
												Nonplastic Silty Sands										
Silty sands (nonplastic)	2-17.0	DS-2	6	17.0		37		NP	92.7	94.5	4.5	S	3.9	27.6	12	0.582	36	3.0	0.763	96.8	2.67	
	2-17.0R	DS-2	6	17.0	Remolded specimen			NP	84.1	87.0 (est)	4.5	L	3.5	0.5	9	0.713	46	0.6	0.915	99.0	2.67	
	1-34.9	BEQ-1U	12	34.9	0.15	35	12	NP	101.8	104.2	44.0	S	143.8*	8.5	25	0.590	36	17.0	0.599	98.3	2.67	
	33.6/34.9	BEQ-1U	12	33.6/ 34.9	Remolded specimen			NP	93.3	95.5	44.0	L	21.1	0.7	2	0.590	32	4.0	0.745	99.5	2.67	
	2-49.9	DS-2	12	4.9		21		NP	99.9	100.4	13.3	S	67.3*	8.0	19	0.582	36	7.1	0.661	99.4	2.67	
	2.64.9	DS-2	22	64.9		18		NP	99.2	99.9	17.3	S	139.4	6.8	20	0.593	36	10.3	0.668	96+	2.67	
	2-64.9R	DS-2	22	64.9	Remolded specimen			NP	87.8	91.5	17.3	LL	9.4	0.8	9	0.588	36	3.1	0.821	99.0	2.67	
												Plastic Sands										
Plastic soils	3-21.4	DS-3	8	21.4	0.023	85	22	10	101.2	102.0	5.7	Plastic	17.2	2.6	9	0.620	38	5.3	0.653	98.3	2.70	7.0
	1-24.9	BEQ-1U	9	24.9	0.014	85	27	7	95.6	98.4	44.0	Plastic	38.3	9.0	8	0.542	33	15.7	0.712	97.5	2.70	18.0
	1-25.7	BEQ-1U	9	25.7/	Remolded specimen			--	95.9	102.0	44.0	Plastic	50.5	16.0	18	0.617	38	12.6	0.652	98.2	2.70	18.0
	25.7/24.9	BEQ-1U	9	24.9 34.9					90.6	107.5	44.0	Plastic	28.4	14.3	12	0.730	47	5.2	0.567	97.5	2.70	12.0
	2-34.9	DS-2	12	61.9		53		10	98.6	99.5	9.3	Plastic	28.3	20.2	23 01705		45	5.9	0.694	98.7	2.70	
	2-61.9	DS-2	21					6	105.8	106.5	16.5	Plastic	45.1	26.2	13	0.571	35	8.4	0.583	96.5	2.70	

* Discontinued loading due to dilative response.

Table 16

Cyclic Triaxial Test Results

Boring	Sample	Depth ft	Effective Over-burden Pressure* psi	D_{50} mm	Percent Fines -74µm	Percent -5µm	Plasticity Index	Void Ratio After Consolidation	Initial Dry Density pcf	As Tested Dry Density pcf	σ_{1c} and σ_{3c} psi	σ_{dc} psi	Stress Ratio SR	Cycles to Double Amplitude Strain N_s	N_{10}	N_{20}	Pore Pressure Response Cycles for Percentages of Effective Stress Reduction 25	50	75	100
									Sands											
BEQ-1U	22	63.4		0.26	2	TR	NP		98.2	99.7	35.0	22.4*	0.320	1.5	**	**	1.0	1.0	1.5	--
BEQ-2U	22	63.9		0.26	5	TR	NP		93.2	94.4	44.0	26.5	0.301	5.0	9.5	**	0.5	1.0	1.5	6.5
BEQ-1U	21	60.1		0.26	9	5	NP		91.5	02.6	44.0	18.9	0.215	3.0	**	**	1.0	2.0	2.0	4.0
									Nonplastic Silty Sands											
BEQ-1U	13	37.6	15	0.11	33	8	NP	0.56	106.7	106.7	9.0	5.02*	0.279	64.5	93.5	†	4.5	34.5	50.5	--
BEQ-1U	13	37.0	15	0.18	27	8	NP	0.56	105.8	106.3	9.0	5.44*	0.302	17.0	35.5	109.0	2.0	4.5	7.5	12.0
BEQ-1U	13	36.4	15	0.18	24	7	NP	0.61	103.4	103.8	9.0	6.01	0.334	14.0	20.0	27.5	1.5	4.5	8.5	13.5
BEQ-1U	14	39.6	16	0.13	33	11	NP	0.65	100.3	101.0	9.0	5.25	0.292	16.5	22.0	40.0	1.5	5.5	7.5	14.5
BEQ-1U	7	18.7	7	0.11	24	7	NP	0.80	91.7	92.4	9.0	5.27	0.293	7.0	9.0	15.0	1.5	3.5	4.5	6.5
BEQ-2U	7	19.7	7	0.10	30	10	NP	0.71	96.2	97.7	9.0	6.41	0.356	4.0	6.5	15.5	1.0	1.0	1.5	2.5
BEQ-2U	7	19.1	7	0.10	28	7	NP	0.76	94.5	94.7	9.0	4.80	0.278	13.5	16.5	29.5	1.5	4.5	7.5	11.5
BEQ-2U	16	45.5	18	0.15	19	7	NP	0.65	100.4	100.9	9.0	6.64	0.369	8.0	12.0	37.5	0.5	2.5	4.0	6.5
BEQ-2U	17	49.9	20	0.13	32	10	NP	0.64	99.8	101.9	9.0	6.47	0.359	7.0	10.0	18.5	0.5	1.5	2.5	5.5
BEQ-1U	16	46.3	18	0.16	19	7	NP	0.63	101.1	102.5	44.0	17.3	0.197	18.5	37.0	113.0	1.5	5.5	10.5	16.5
BEQ-1U	12	33.6	13	0.21	19	7	NP	0.60	101.4	103.9	44.0	20.1	0.228	8.5	11.0	19.0	1.5	3.0	4.5	8.0
BEQ-2U	14	39.9	16	0.13	33	10	NP	0.56	105.1	106.7	44.0	25.6	0.291	4.0	6.5	16.0	1.0	1.0	1.5	3.5
BEQ-2U	16	46.1	18	0.13	25	10	NP	0.66	96.8	100.1	44.0	26.2	0.297	2.5	4.5	9.0	0.5	1.0	1.5	3.5
BEQ-2U	16	46.7	18	0.13	27	10	NP	0.63	100.0	102.0	44.0	21.4	0.243	6.0	9.0	2.0	0.5	2.0	3.0	5.5
BEQ-2U	22	63.3	25	0.20	16	8	NP	0.68	96.9	99.1	44.0	20.2	0.229	7.5	11.0	19.0	1.5	2.5	4.0	8.5
BEQ-2U	21	60.5	24	0.22	24	12	NP	0.66	97.7	100.5	44.0	19.7	0.224	8.0	12.5	15.0	1.5	2.5	5.0	7.5
BEQ-2U	23	67.7	26	0.21	27	12	NP	0.62	101.0	103.1	44.0	19.4	0.221	9.5	14.0	23.0	0.5	2.5	4.5	11.5
									Specimens With Plastic Fines											
BEQ-1U	14	40.2		0.073	5	16	--		100.3	101.9	9.0	4.67	0.259	36.0	42.5	64.0	1.5	10.5	22.5	29.5
BEQ-1U	7	19.9		0.082	46	15	9		99.6	101.2	9.0	6.02	0.334	10.0	21.5	56.0	0.5	0.5	2.5	4.5
BEQ-1U	7	19.3		0.080	43	9	7		96.3	99.4	9.0	7.04*	0.391	5.0	9.5	29.0	0.5	1.0	1.5	3.5
BEQ-1U	6	16.9		0.035	67	20	8		105.8	108.2	44.0	22.4	0.255	2.5	3.5	4.0	0.5	1.5	2.5	3.5
BEQ-1U	6	16.3		0.012	89	32	13		102.3	106.4	44.0	19.7	0.224	8.0	8.5	9.5	2.5	4.5	6.5	8.5
BEQ-1U	27	77.1		0.015	79	29	8		94.8	98.4	44.0	20.2	0.229	30.5	53.5	60.5**	1.5	3.5	8.5	29.5

* Load wave not symmetrical.
** Specimen necked.
† Load fell off.

Table 17

Shear Strength Results - BEQ 3U-8U

Material	Natural Water Content %	Dry Unit Weight pcf	Moist Unit Weight pcf	Atterberg Limits		Consolidated-Undrained Shear Strength (R)		Drained Shear Strength (S)		Effective Friction Angle
				LL	PL	c (tsf)	tan ϕ	c (tsf)	tan ϕ	ϕ'
Embankment, Switchyard	16.2	114.2	132.7	36	19	0.45	0.503	0	0.617	31.67°
CL (Unit 1)	22.4	103.1	126.0	38	20	0.57	0.279	0	0.653	33.14°
CL (Unit 2)	24.0	99.8	123.5	33	19	0.54	0.233	0	0.640	32.62°

Table 18

Barkley Dam (Tubes)

Sample No.	Depth, ft	Borehole	Natural Water Content Percent	Dry Unit Weight pcf	Penetration Resistance* tsf	Laboratory Vane Shear tsf
1-A	9.8-11.8	BEQ-3U			4.75	16.20
1-B	9.8-11.8	BEQ-3U			3.4	10.25
2-B-C	19.6-21.6	BEQ-3U			4.75	16.50
3-B	29.6-31.6	BEQ-3U	21.8	113.0	1.6	4.0
5-B&C	45.6-47.6	BEQ-3U			1.4	8.2
1-B	15.0-17.5	BEQ-4U			4.75	16.2
2-B	22.0-24.5	BEQ-4U			4.75	15.6
2-C	" "	" "			3.25	15.0
3-B	35.0-37.5	BEQ-4U	20.9	103.7	4.75	10.0
3-C	" "	" "			4.75	14.0
4-C	40.0-42.5	BEQ-4U	21.4	101.3	4.30	10.4
5-B	43.0-42.5	BEQ-4U			3.40	7.40
5-C	" "	" "			3.50	7.90
6-C	46.5-48.5	BEQ-4U	23.3	99.7	2.60	4.50
6-D	" "	" "			1.50	4.80
7-B	49.0-51.5	BEQ-4U			1.50	5.20
7-D	" "	" "			1.30	4.25
8-B	52.0-54.5	BEQ-4U	23.0	102.9	1.75	7.80
9-C	55.0-57.5	BEQ-4U	21.9	101.7	1.50	2.80
10-C	58.5-60.5	BEQ-4U			1.25	1.60
10-D	" "	" "			2.20	2.50
12-A	68.0-70.5	BEQ-4U	30.7	92.0	0.70	3.10
12-C	" "	" "	25.6	117.1	0.50	1.30
2-A	12.0-13.9	BEQ-5U	19.7	106.2	0.85	4.10
22-C	10.0-12.5	BEQ-6U	26.4	97.5	2.00	4.60
22-B	" "	" "	11.6	101.3		
2-B	20.0-21.85	BEQ-7U	22.1	100.7	3.20	13.0
4-B	22.7-24.52	BEQ-7U	20.3	102.4	4.25	10.0
6-B	27.5-29.5	BEQ-7U	20.3	102.1	2.50	8.20
7-B	30.0-32.0	BEQ-7U	21.7	98.6	1.50	5.20
9-A	32.5-34.5	BEQ-7U	20.6	121.2	1.80	4.50
11-B	37.5-39.5	BEQ-7U	27.3	129.8	3.10	6.30
12-B	40.0-42.0	BEQ-7U	21.9	100.8		
13-A	42.5-44.5	BEQ-7U	25.5	97.4	1.25	5.50
16-A	45.0-47.0	BEQ-7U	30.4	91.2		
18-B	50.0-52.0	BEQ-7U	38.0	84.1	1.25	6.00
19-A	57.0-59.5	BEQ-7U	26.4	94.9	1.75	5.00
19-B	" "	" "	20.0	103.2	2.90	4.00
21-D	74.5-77.0	BEQ-7U			1.80	4.30

* Pocket penetrometer.

Table 19

Measured and Estimated In Situ Steady-State Shear Strengths and Void Ratios of Foundation Sand

				In Situ Values Estimated from Laboratory Values Corrected for Sample Volume Change			
		Values Measured In Laboratory		Assumption: Uniform Volume Change In Tube Sample		Assumption: Sand Undergoes All Measured Compression But No Expansion	
Test No.	Material Group (% Fines)	Void Ratio e	Steady-State Shear Strength S_{us} psi	Void Ratio e	Steady-State Shear Strength S_{us} psi	Void Ratio e	Steady-State Shear Strength S_{us} psi
R-4	12-16	0.749	66	0.776	46	0.805	28
R-6	12-16	0.684	99	0.742	45	0.741	43
R-8	12-16	0.733	133	0.752	105	0.761	95
R-9	12-16	0.680	129	0.726	77	0.737	66
R-13	12-16	0.618	40	0.667	17	0.680	13
R-1	18-44	0.721	26	0.746	20	0.755	14
R-3	18-44	0.617	35	0.677	9	0.692	6
R-5	18-44	0.630	71	0.703	20	0.703	20
R-7	18-44	0.692	64	0.780	15	0.790	11
R-10	18-44	0.548	15	0.579	9	0.598	6
R-11	18-44	0.730	62	0.784	29	0.794	22
R-12	18-44	0.759	55	0.875	7	0.894	5

Table 20

CPT Locations and Testing Program

CPT No.	Location L	Location B	Top of Ground Elevation	Instrument No.*	Data Measurements (ft) Depth of q_c & f_s	Pore Pressure	Conductivity	Dielectric
1	38+71	0+65	378.5	080	97.1	--	--	--
2	38+71	1+10	365.7	080	81.9	--	--	--
3	35+54	1+52	365.7	070,076	81.7	50.2-81.7	81.7	--
4	36+05	1+50	365.7	070	83.2	--	83.2	--
5	36+45	1+50	365.8	076	79.7	--	79.7	--
6	37+05	1+50	365.8	070,076	86.6	34.0-86.6	--	--
7	37+61	1+50	365.8	076	83.1	--	83.1	--
8	38+17	1+50	365.7	229,070	61.0	--	62.4	--
9	38+71	1+50	365.8	070,076	79.9	30.0-79.9	79.9	--
10	39+27	1+50	365.8	076	83.1	--	83.1	--
11	39+85	1+60	365.2	076	83.0	--	83.0	--
12	38+70	2+07	365.7	080	86.6	--	--	--
13	39+85	2+15	363.6	080	79.9	--	--	--
14	38+55	2+68	365.5	070,076	82.3	35.1-82.3	82.3	--
15	34+94	2+81	365.5	076	83.1	--	83.1	--
16	38+41	3+13	364.7	080	79.2	--	--	--
17	39+85	3+20	361.7	080	78.9	--	--	--
18	34+94	3+41	365.6	076	80.1	--	80.1	--
19	38+34	3+57	364.7	070,076	78.9	44.8-78.9	78.9	--
20	39+35	3+70	361.4	080	79.0	--	--	--
21	34+52	4+51	340.7	080	58.6	--	--	--
22	38+07	4+22	348.4	080	65.1	--	--	--
23	39+85	4+30	350.5	080	68.8	--	--	--
24	34+46	4+65	341.1	076	60.0	--	60.0	--
25	34+46	4+65	341.5	229	59.7	--	--	--
26	34+56	4+98	341.5	080	67.1	--	--	--
27	34+97	4+91	342.2	080	43.4	--	--	--
28	35+29	4+91	343.4	070,080	59.8	30.0-59.8	59.8	--
29	35+61	4+92	344.8	070	59.9	--	--	--
30	35+93	4+93	345.9	229	46.5	--	--	--

(Continued)

* Instruments:
080 subtracting design probe.
229 tension design probe.
070 subtracting probe with conductivity unit.
076 subtracting probe with conductivity unit and piezo element.

Table 20 (Concluded)

CPT No.	Location L	Location B	Top of Ground Elevation	Instrument No.	Data Measurements (ft) Depth of q_c & f_s	Pore Pressure	Conductivity	Dielectric
31	36+25	4+96	346.6	076-080	49.6	35.1-49.7	--	--
32	36+55	4+98	346.9	229	50.1	--	--	--
33	36+83	5+00	347.2	076	66.8	--	--	--
34	37+85	5+00	348.0	070,076.080	65.8	34.9-65.8	65.9	--
35	38+70	5+00	349.9	076	66.3	--	66.3	--
36	39+50	4+90	349.8	070,076	67.7	30.5-67.7	67.7	--
37	40+00	4+96	350.4	076	73.1	--	73.1	--
38	40+50	5+00	350.9	080	69.1	--	--	--
39	41+50	5+00	351.7	076	76.5	--	76.5	--
40	42+50	5+00	352.8	080	85.0	--	--	--
41	34+38	5+03	341.7	070,076	56.9	29.8-56.9	56.9	--
42	34+35	5+37	341.8	076	63.1	--	63.1	--
43	34+35	5+68	342.0	229,076	44.2	--	59.9	--
44	37+61	5+53	348.0	076	69.6	--	69.6	--
45	40+00	5+48	350.0	080	69.8	--	--	--
46	34+35	6+00	342.2	070	59.7	--	59.7	--
47	34+35	6+30	342.7	080	62.3	--	--	--
48	37+32	6+35	347.5	076,080	69.1	34.9-69.1	69.1	--
49	40+00	6+22	349.8	080	64.1	--	--	--
50	34+35	6+60	342.8	070	46.9	--	46.9	--
51	34+35	6+90	342.7	070,076	47.2	--	46.6	--
52	37+05	7+00	347.1	076	60.0	--	60.0	--
53	40+00	6+90	347.8	070,076,080	69.4	25.0-69.4	69.9	--
54	34+71	4+92	341.5	070,076	58.3	30.1-58.3	58.3	3.1-42.7
55	34+68	4+85	341.5	229	44.1	--	--	--
56	34+58	4+76	341.1	076	66.7	--	66.7	0.5-41.0
57	34+51	4+71	341.1	070	59.0	--	59.0	--
58	33+62	5+04	341.9	229	43.4	--	--	--
65	39+92	2+69	362.5	080	76.3	--	--	--

Table 21

CPT Data from Barkley Dam Foundation

"Earthquake=",8.5

Name	Elevation	EQfile	No.	Boring1	Boring2	Boring3	Boring4	Boring5
"CPT-01",	378.5,	"SW",	0,	"",	"",	"",	"",	""
"CPT-02",	365.7,	"SW",	0,	"",	"",	"",	"",	""
"CPT-03",	365.7,	"SW",	2,	"BEQ-15",	"BEQ-16",	"",	"",	""
"CPT-04",	365.7,	"SW",	0,	"",	"",	"",	"",	""
"CPT-05",	365.8,	"SW",	0,	"",	"",	"",	"",	""
"CPT-06",	365.8,	"SW",	2,	"BEQ-17",	"BEQ-18",	"",	"",	""
"CPT-07",	365.8,	"SW",	0,	"",	"",	"",	"",	""
"CPT-08",	365.7,	"SW",	0,	"",	"",	"",	"",	""
"CPT-09",	365.8,	"SW",	0,	"",	"",	"",	"",	""
"CPT-10",	365.8,	"SW",	0,	"",	"",	"",	"",	""
"CPT-11",	365.2,	"SW",	2,	"BEQ-19",	"BEQ-20",	"",	"",	""
"CPT-12",	363.6,	"SW",	0,	"",	"",	"",	"",	""
"CPT-13",	363.6,	"SW",	0,	"",	"",	"",	"",	""
"CPT-14",	365.5,	"SW",	0,	"",	"",	"",	"",	""
"CPT-15",	365.5,	"SW",	0,	"",	"",	"",	"",	""
"CPT-16",	364.7,	"SW",	0,	"",	"",	"",	"",	""
"CPT-17",	361.7,	"SW",	0,	"",	"",	"",	"",	""
"CPT-18",	365.7,	"SW",	0,	"",	"",	"",	"",	""
"CPT-19",	364.7,	"SW",	0,	"",	"",	"",	"",	""
"CPT-20",	361.4,	"SW",	0,	"",	"",	"",	"",	""
"CPT-21",	340.7,	"FF",	0,	"",	"",	"",	"",	""
"CPT-22",	348.4,	"FF",	0,	"",	"",	"",	"",	""
"CPT-23",	350.5,	"FF",	0,	"",	"",	"",	"",	""
"CPT-24",	341.1,	"FF",	0,	"",	"",	"",	"",	""
"CPT-25",	341.5,	"FF",	3,	"BEQ-21",	"BEQ-22",	"DS-3",	"BEQ-8U",	""
"CPT-26",	341.5,	"FF",	0,	"",	"",	"",	"",	""
"CPT-27",	342.2,	"FF",	0,	"",	"",	"",	"",	""
"CPT-28",	343.4,	"FF",	0,	"",	"",	"",	"",	""
"CPT-29",	344.8,	"FF",	0,	"",	"",	"",	"",	""
"CPT-30",	345.9,	"FF",	0,	"",	"",	"",	"",	""
"CPT-31",	346.6,	"FF",	0,	"",	"",	"",	"",	""
"CPT-32",	346.9,	"FF",	0,	"",	"",	"",	"",	""
"CPT-33",	347.2,	"FF",	2,	"BEQ-23",	"BEQ-24",	"",	"",	""
"CPT-34",	348.0,	"FF",	0,	"",	"",	"",	"",	""
"CPT-35",	349.9,	"FF",	0,	"",	"",	"",	"",	""

(Continued)

Table 21 (Concluded)

"Earthquake=",8.5

Name	Elevation	EQfile	No.	Boring1	Boring2	Boring3	Boring4	Boring5
"CPT-36",	349.8,	"FF",	3,	"BEQ-08",	"BEQ-25",	"BEQ-26",	"",	""
"CPT-37",	350.4,	"FF",	0,	"",	"",	"",	"",	""
"CPT-38",	350.9,	"FF",	0,	"",	"",	"",	"",	""
"CPT-39",	351.7,	"FF",	0,	"",	"",	"",	"",	""
"CPT-40",	352.8,	"FF",	0,	"",	"",	"",	"",	""
"CPT-41",	341.7,	"FF",	3,	"BEQ-22",	"BEQ-21",	"BEQ-07",	"",	""
"CPT-42",	341.8,	"FF",	0,	"",	"",	"",	"",	""
"CPT-43",	342.0,	"FF",	0,	"",	"",	"",	"",	""
"CPT-44",	348.0,	"FF",	0,	"",	"",	"",	"",	""
"CPT-45",	350.0,	"FF",	0,	"",	"",	"",	"",	""
"CPT-46",	342.2,	"FF",	0,	"",	"",	"",	"",	""
"CPT-47",	342.7,	"FF",	0,	"",	"",	"",	"",	""
"CPT-48",	347.5,	"FF",	0,	"",	"",	"",	"",	""
"CPT-49",	349.8,	"FF",	0,	"",	"",	"",	"",	""
"CPT-50",	342.8,	"FF",	0,	"",	"",	"",	"",	""
"CPT-51",	342.7,	"FF",	2,	"BEQ-27",	"BEQ-28",	"",	"",	""
"CPT-52",	347.1,	"FF",	2,	"BEQ-30",	"BEQ-29",	"",	"",	""
"CPT-53",	347.8,	"FF",	2,	"BEQ-31",	"BEQ-32",	"",	"",	""
"CPT-54",	341.5,	"FF",	0,	"",	"",	"",	"",	""
"CPT-55",	341.5,	"FF",	0,	"",	"",	"",	"",	""
"CPT-56",	341.1,	"FF",	0,	"",	"",	"",	"",	""
"CPT-57",	341.1,	"FF",	0,	"",	"",	"",	"",	""
"CPT-58",	341.9,	"SW",	0,	"",	"",	"",	"",	""
"CPT-59",	356.7,	"SW",	0,	"",	"",	"",	"",	""
"CPT-60",	365.7,	"SW",	0,	"",	"",	"",	"",	""
"CPT-61",	365.8,	"SW",	0,	"",	"",	"",	"",	""
"CPT-62",	365.9,	"SW",	0,	"",	"",	"",	"",	""
"CPT-63",	365.7,	"SW",	0,	"",	"",	"",	"",	""
"CPT-64",	365.8,	"SW",	0,	"",	"",	"",	"",	""
"CPT-65",	362.5,	"SW",	2,	"BEQ-33",	"BEQ-34",	"",	"",	""

Table 22

Borings Near CPT Soundings at Barkley Dam

"Earthquake=", 8.5

Name	Elevation	EQfile	No.	CPT1	CPT2	CPT3	CPT4	CPT5
"BEQ-01",	378.5,	"FF",	0	"",	"",	"",	"",	"",
"BEQ-02",	365.7,	"FF",	0	"",	"",	"",	"",	"",
"BEQ-03",	365.7,	"FF",	2	"",	"",	"",	"",	"",
"BEQ-04",	365.7,	"FF",	0	"",	"",	"",	"",	"",
"BEQ-05",	365.8,	"FF",	0	"",	"",	"",	"",	"",
"BEQ-06",	365.8,	"FF",	0	"",	"",	"",	"",	"",
"BEQ-07",	365.8,	"FF",	1	"CPT-25",	"",	"",	"",	"",
"BEQ-08",	365.7,	"FF",	0	"",	"",	"",	"",	"",
"BEQ-09",	365.8,	"FF",	0	"",	"",	"",	"",	"",
"BEQ-10",	365.8,	"FF",	0	"",	"",	"",	"",	"",
"BEQ-11",	365.2,	"FF",	0	"",	"",	"",	"",	"",
"BEQ-12",	363.6,	"FF",	0	"",	"",	"",	"",	"",
"BEQ-13",	363.6,	"FF",	0	"",	"",	"",	"",	"",
"BEQ-14",	365.5,	"FF",	0	"",	"",	"",	"",	"",
"BEQ-15",	365.5,	"SW",	1	"CPT-03",	"",	"",	"",	"",
"BEQ-16",	364.7,	"SW",	1	"CPT-03",	"",	"",	"",	"",
"BEQ-17",	361.7,	"SW",	1	"CPT-06",	"",	"",	"",	"",
"BEQ-18",	365.7,	"SW",	1	"CPT-06",	"",	"",	"",	"",
"BEQ-19",	364.7,	"SW",	1	"CPT-11",	"",	"",	"",	"",
"BEQ-20",	361.4,	"SW",	1	"CPT-11",	"",	"",	"",	"",
"BEQ-21",	340.7,	"FF",	2	"CPT-25",	"CPT-41",	"",	"",	"",
"BEQ-22",	348.4,	"FF",	2	"CPT-25",	"CPT-41",	"",	"",	"",
"BEQ-23",	350.5,	"FF",	1	"CPT-33",	"",	"",	"",	"",
"BEQ-24",	341.1,	"FF",	1	"CPT-33",	"",	"",	"",	"",
"BEQ-25",	341.5,	"FF",	1	"CPT-36",	"",	"",	"",	"",
"BEQ-26",	341.5,	"FF",	1	"CPT-36",	"",	"",	"",	"",
"BEQ-27",	342.2,	"FF",	1	"CPT-51",	"",	"",	"",	"",
"BEQ-28",	343.4,	"FF",	1	"CPT-51",	"",	"",	"",	"",
"BEQ-29",	344.8,	"FF",	1	"CPT-52",	"",	"",	"",	"",
"BEQ-30",	345.9,	"FF",	1	"CPT-52",	"",	"",	"",	"",

(Continued)

Table 22 (Concluded)

Name	Elevation	EQfile	No.	CPT1	CPT2	CPT3	CPT4	CPT5
"BEQ-31",	346.6,	"FF",	1	"CPT-53",	"",	"",	"",	"",
"BEQ-32",	346.9,	"FF",	1	"CPT-53",	"",	"",	"",	"",
"BEQ-33",	346.6,	"SW",	1	"CPT-65",	"",	"",	"",	"",
"BEQ-34",	346.9,	"SW",	1	"CPT-65",	"",	"",	"",	"",
"BEQ-5U",	346.6,	"FF",	2	"CPT-55",	"CPT-56",	"",	"",	"",
"BEQ-6U",	346.9,	"FF",	1	"CPT-54",	"CPT-58",	"",	"",	"",
"BEQ-8U",	346.9,	"FF",	3	"CPT-57",	"CPT-24",	"CPT-25",	"",	"",
"DS-1",	341.8,	"FF",	0	"",	"",	"",	"",	"",
"DS-2",	342.0,	"FF",	0	"",	"",	"",	"",	"",
"DS-3",	348.0,	"FF",	1	"CPT-25",	"",	"",	"",	"",
"B-D-1",	342.0,	"FF",	0	"",	"",	"",	"",	"",
"B-D-2",	342.0,	"FF",	0	"",	"",	"",	"",	"",
"B-D-3",	342.0,	"FF",	0	"",	"",	"",	"",	"",
"B-D-4",	342.0,	"FF",	0	"",	"",	"",	"",	"",
"B-D-5",	342.0,	"FF",	0	"",	"",	"",	"",	"",
"B-D-6",	342.0,	"FF",	0	"",	"",	"",	"",	"",
"B-D-7",	342.0,	"FF",	0	"",	"",	"",	"",	"",
"B-D-8",	342.0,	"FF",	0	"",	"",	"",	"",	"",
"B-D-9",	342.0,	"FF",	0	"",	"",	"",	"",	"",

Table 23

CPT-1 Analysis of Barkley Dam

DATE OF TEST : 05/22/85
INSTRUMENT ID : F15CKE080
WATER TABLE : 38.000
CONE SMOOTH : 0.000
FRIC SMOOTH : 0.000
PORE SMOOTH : 0.000
COND SMOOTH : 0.000
NO OF DATA PTS: 974
CONE-FRIC LEAD: 0.350
CONE-PORE LEAD: 0.010
CONE-COND LEAD: 0.900
GAMMA OF WATER: 62.400
GAMMA ABOVE WT: 123.000
GAMMA BELOW WT: 125.000
RF SMOOTH : 0.500
RU SMOOTH : 0.000
RC SMOOTH : 0.000

0.0	0.01	0.01	0.00	0.0	0.00
0.1	0.00	0.05	0.00	0.0	20.00
0.2	0.00	0.05	0.00	0.0	16.00
0.3	0.00	0.04	0.00	0.0	20.00
0.4	0.00	0.05	0.00	0.0	20.00
0.5	0.00	0.06	0.00	0.0	18.10
0.6	0.00	0.24	0.00	0.0	14.30
0.7	3.33	0.36	0.00	0.0	10.50
0.8	40.82	0.37	0.00	0.0	6.80
0.9	78.10	0.80	0.00	0.0	3.40
1.0	84.76	1.38	0.00	0.0	1.90
1.1	64.93	1.70	0.00	0.0	2.90
1.2	49.55	1.88	0.00	0.0	3.70
1.3	35.79	2.07	0.00	0.0	4.20
1.4	44.72	2.20	0.00	0.0	4.70
1.5	60.91	2.55	0.00	0.0	5.30
1.6	59.55	2.67	0.00	0.0	6.20
1.7	39.60	2.62	0.00	0.0	6.50
1.8	29.16	2.99	0.00	0.0	6.70
1.9	54.98	3.52	0.00	0.0	6.40
2.0	80.80	4.00	0.00	0.0	6.10
2.1	105.17	4.46	0.00	0.0	5.20
2.2	96.44	4.91	0.00	0.0	5.10
2.3	85.12	4.74	0.00	0.0	5.40
2.4	73.80	4.38	0.00	0.0	5.70
2.5	62.48	4.03	0.00	0.0	5.80
2.6	62.17	3.65	0.00	0.0	5.90
2.7	62.04	3.42	0.00	0.0	6.10
2.8	52.44	3.19	0.00	0.0	6.00

(Continued)

Table 23 (Continued)

2.9	44.75	2.96	0.00	0.0	5.90
3.0	40.88	2.61	0.00	0.0	5.90
3.1	49.61	2.52	0.00	0.0	5.90
3.2	45.61	2.55	0.00	0.0	5.90
3.3	40.76	2.52	0.00	0.0	6.10
3.4	36.77	2.44	0.00	0.0	6.50
3.5	33.33	2.34	0.00	0.0	6.90
3.6	30.25	2.23	0.00	0.0	7.10
3.7	29.59	2.18	0.00	0.0	7.10
3.8	30.32	2.15	0.00	0.0	7.10
3.9	30.69	2.15	0.00	0.0	7.00
4.0	30.05	2.05	0.00	0.0	6.80
4.1	29.40	2.00	0.00	0.0	6.60
4.2	28.76	1.84	0.00	0.0	6.30
4.3	27.78	1.69	0.00	0.0	6.10
4.4	26.76	1.57	0.00	0.0	5.80
4.5	26.23	1.45	0.00	0.0	5.70
4.6	26.67	1.45	0.00	0.0	5.70
4.7	26.84	1.58	0.00	0.0	5.80
4.8	26.50	1.59	0.00	0.0	5.90
4.9	26.16	1.69	0.00	0.0	5.90
5.0	25.46	1.53	0.00	0.0	5.50
5.1	24.66	1.27	0.00	0.0	5.30
5.2	28.65	1.23	0.00	0.0	5.10
5.3	32.44	1.48	0.00	0.0	5.40
5.4	30.44	1.70	0.00	0.0	5.80
5.5	24.34	1.83	0.00	0.0	6.30
5.6	22.51	1.69	0.00	0.0	6.50
5.7	25.39	1.62	0.00	0.0	6.60
5.8	27.59	1.64	0.00	0.0	6.40
5.9	29.79	1.78	0.00	0.0	6.20
6.0	30.33	1.96	0.00	0.0	6.10
6.1	29.06	1.85	0.00	0.0	6.00
6.2	29.55	1.75	0.00	0.0	5.90
6.3	29.32	1.66	0.00	0.0	5.70
6.4	29.71	1.54	0.00	0.0	5.60
6.5	29.65	1.66	0.00	0.0	5.70
6.6	30.59	1.81	0.00	0.0	5.90
6.7	32.35	1.98	0.00	0.0	6.20
6.8	34.05	2.29	0.00	0.0	6.40
6.9	36.39	2.44	0.00	0.0	6.40
7.0	37.84	2.47	0.00	0.0	6.40
7.1	39.15	2.48	0.00	0.0	6.20
7.2	40.56	2.40	0.00	0.0	6.10
7.3	41.52	2.30	0.00	0.0	6.30
7.4	36.10	2.33	0.00	0.0	6.60
7.5	30.67	2.26	0.00	0.0	7.00

(Continued)

Table 23 (Concluded)

7.6	25.24	2.04	0.00	0.0	7.20
7.7	23.80	1.80	0.00	0.0	7.20
7.8	26.40	1.82	0.00	0.0	7.10
7.9	30.43	1.97	0.00	0.0	6.90
8.0	31.88	2.12	0.00	0.0	6.90
8.1	31.30	2.21	0.00	0.0	7.00
8.2	29.21	2.17	0.00	0.0	7.30
8.3	27.12	2.09	0.00	0.0	7.60
8.4	25.10	2.01	0.00	0.0	7.80
8.5	23.82	1.95	0.00	0.0	7.80
8.6	25.34	1.97	0.00	0.0	7.70
8.7	26.34	1.96	0.00	0.0	7.70
8.8	27.12	1.90	0.00	0.0	7.60
8.9	27.39	2.18	0.00	0.0	7.50
9.0	28.62	2.20	0.00	0.0	7.40
9.1	31.58	2.29	0.00	0.0	7.20
9.2	39.39	2.71	0.00	0.0	6.90
9.3	45.20	3.01	0.00	0.0	6.60
9.4	46.38	2.94	0.00	0.0	6.40
9.5	45.97	2.81	0.00	0.0	6.20
9.6	42.23	2.57	0.00	0.0	6.20
9.7	37.89	2.25	0.00	0.0	6.50
9.8	30.12	2.04	0.00	0.0	6.90
9.9	24.23	1.89	0.00	0.0	7.30
10.0	21.51	1.74	0.00	0.0	7.40
10.1	19.47	1.60	0.00	0.0	7.20
10.2	23.94	1.48	0.00	0.0	7.00
10.3	24.01	1.42	0.00	0.0	6.90
10.4	21.15	1.42	0.00	0.0	7.10
10.5	18.29	1.45	0.00	0.0	7.70
10.6	16.75	1.48	0.00	0.0	8.60
10.7	15.92	1.51	0.00	0.0	9.10
10.8	15.27	1.56	0.00	0.0	8.90
10.9	17.84	1.63	0.00	0.0	8.10
11.0	24.45	1.72	0.00	0.0	7.20
11.1	35.98	1.82	0.00	0.0	6.30

(Sheet 3 of 3)

Figure 1. Site map

1 of 2

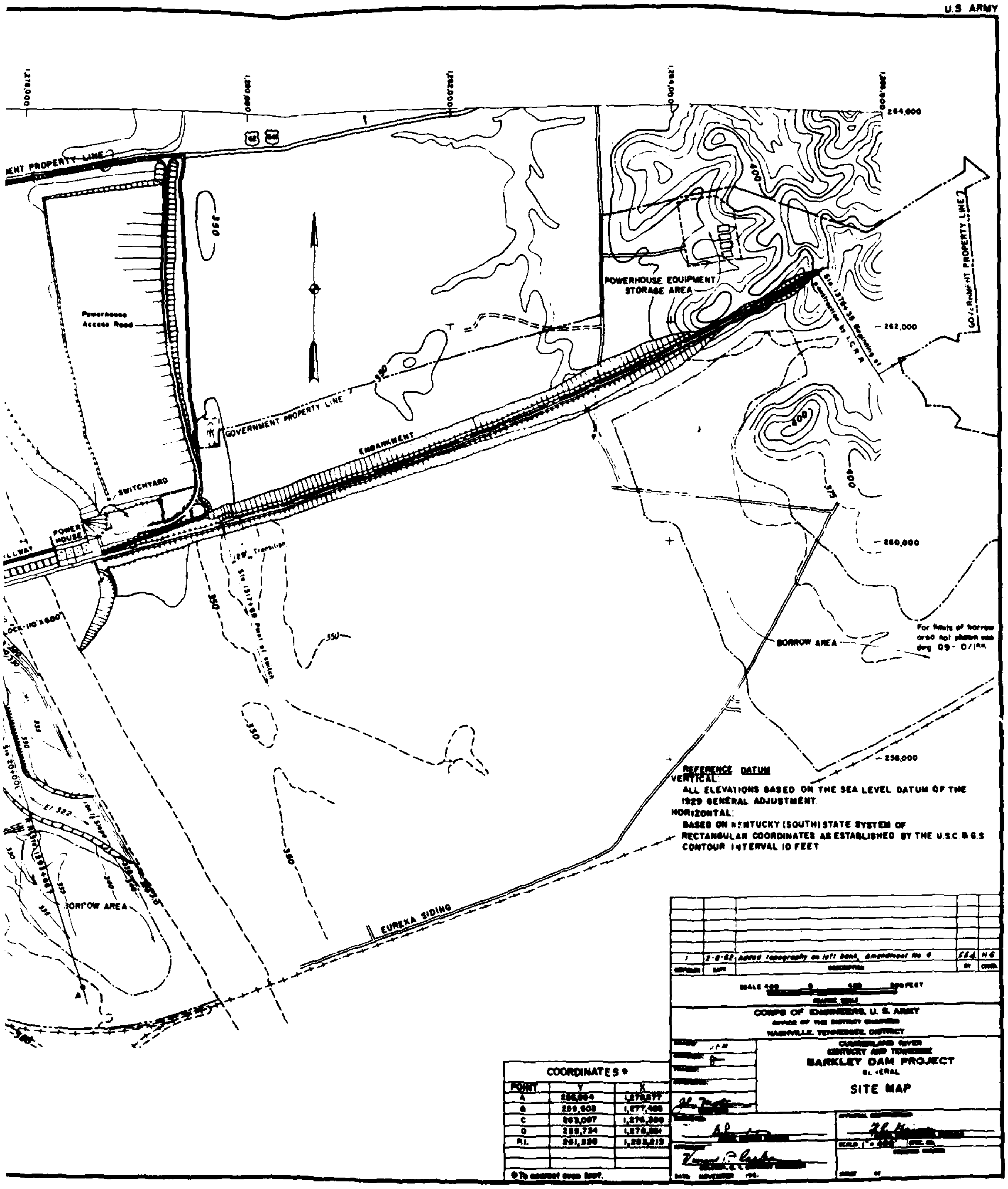

Figure 1. Site map

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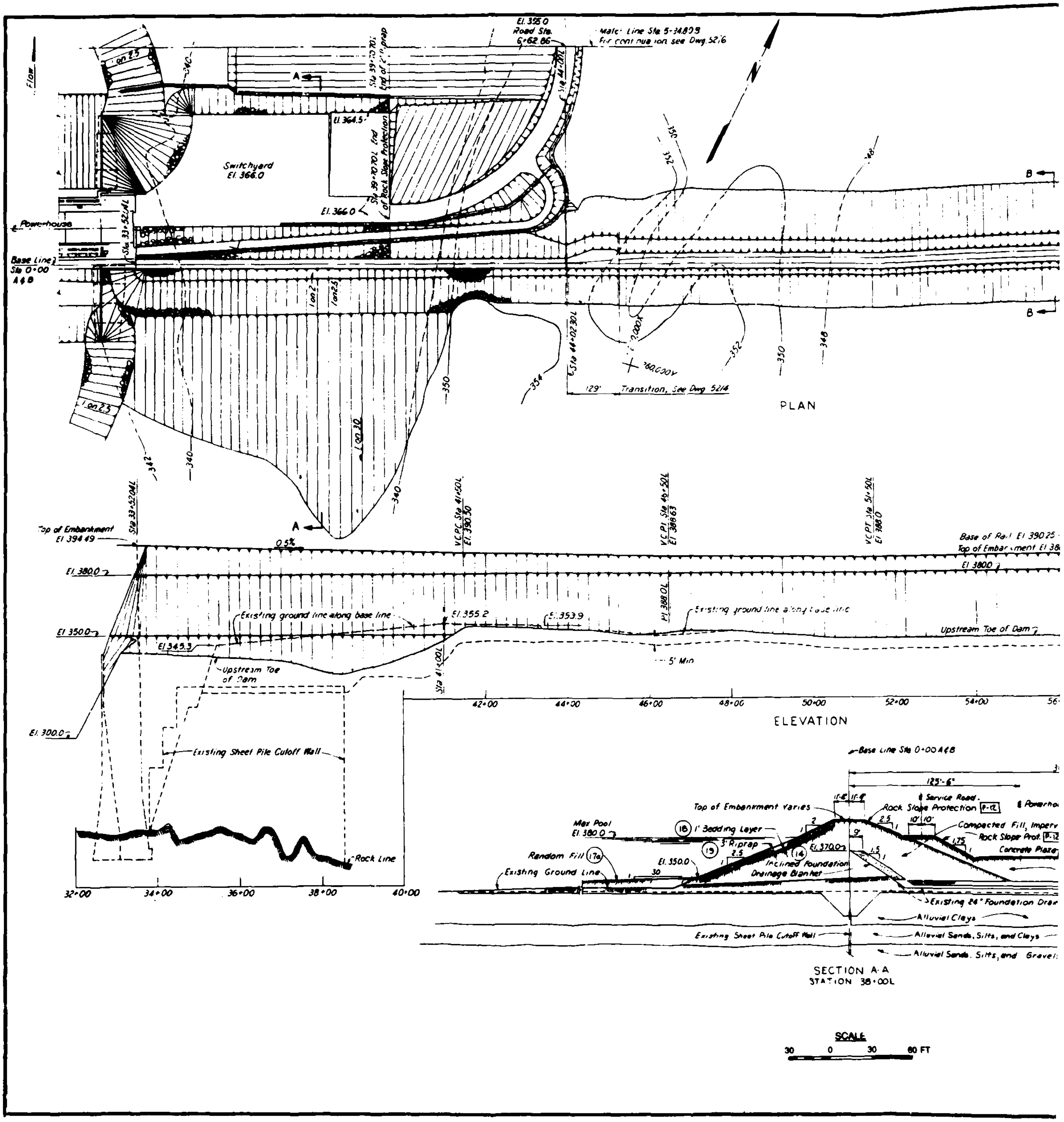
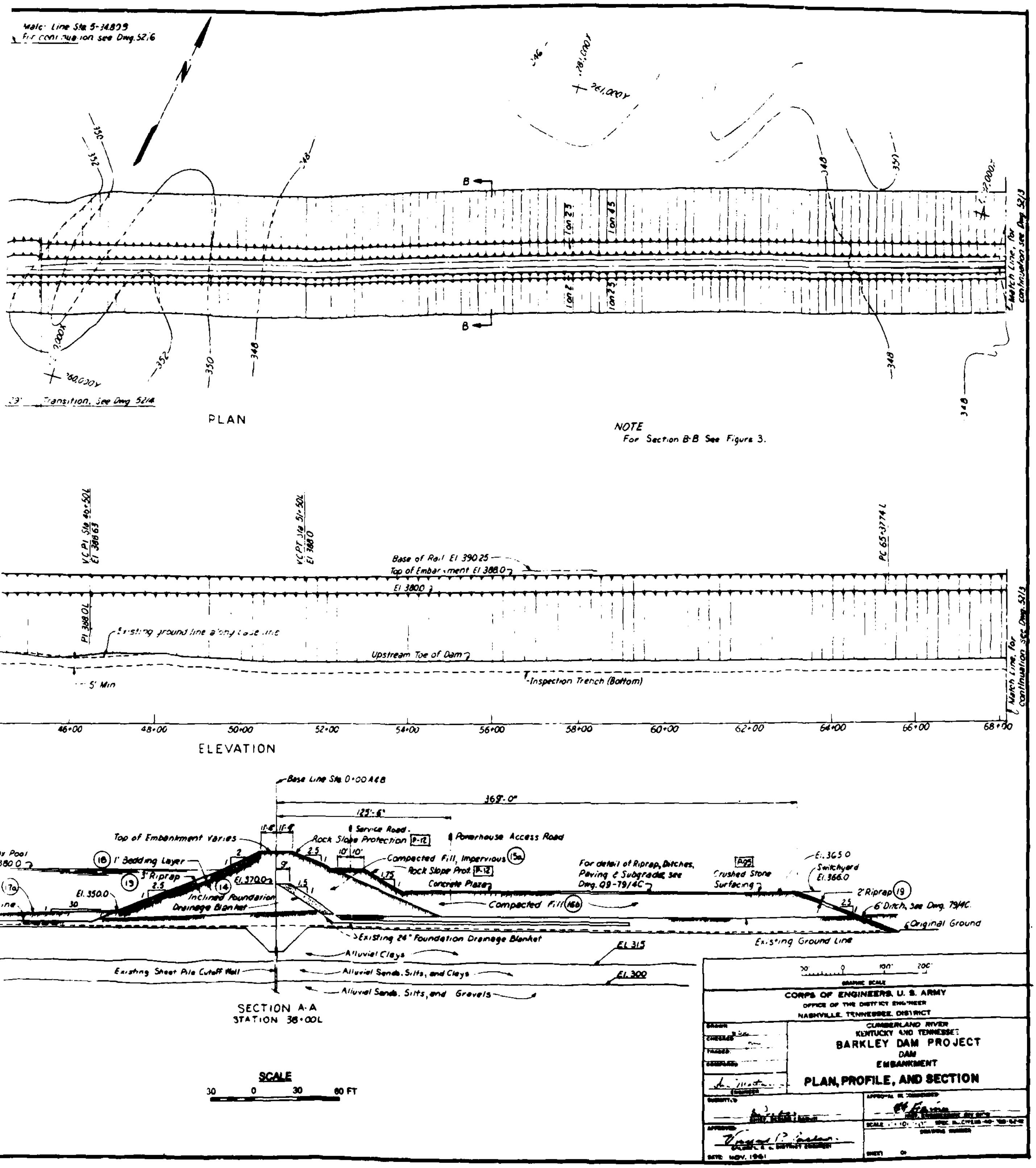


Figure 2. Dam embankment plan, profile and section

1 of 2



e 2. Dam embankment plan, profile and section

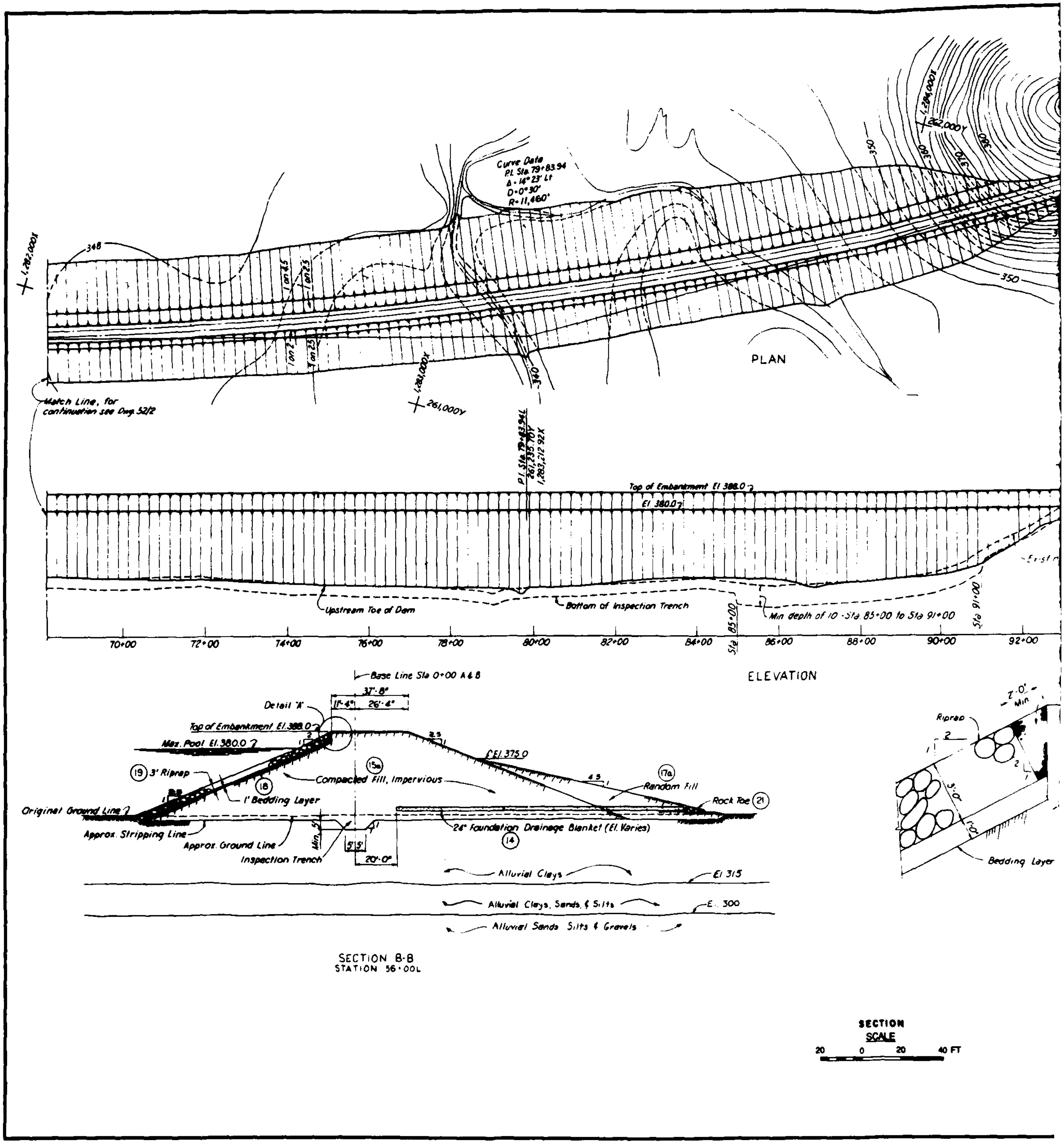
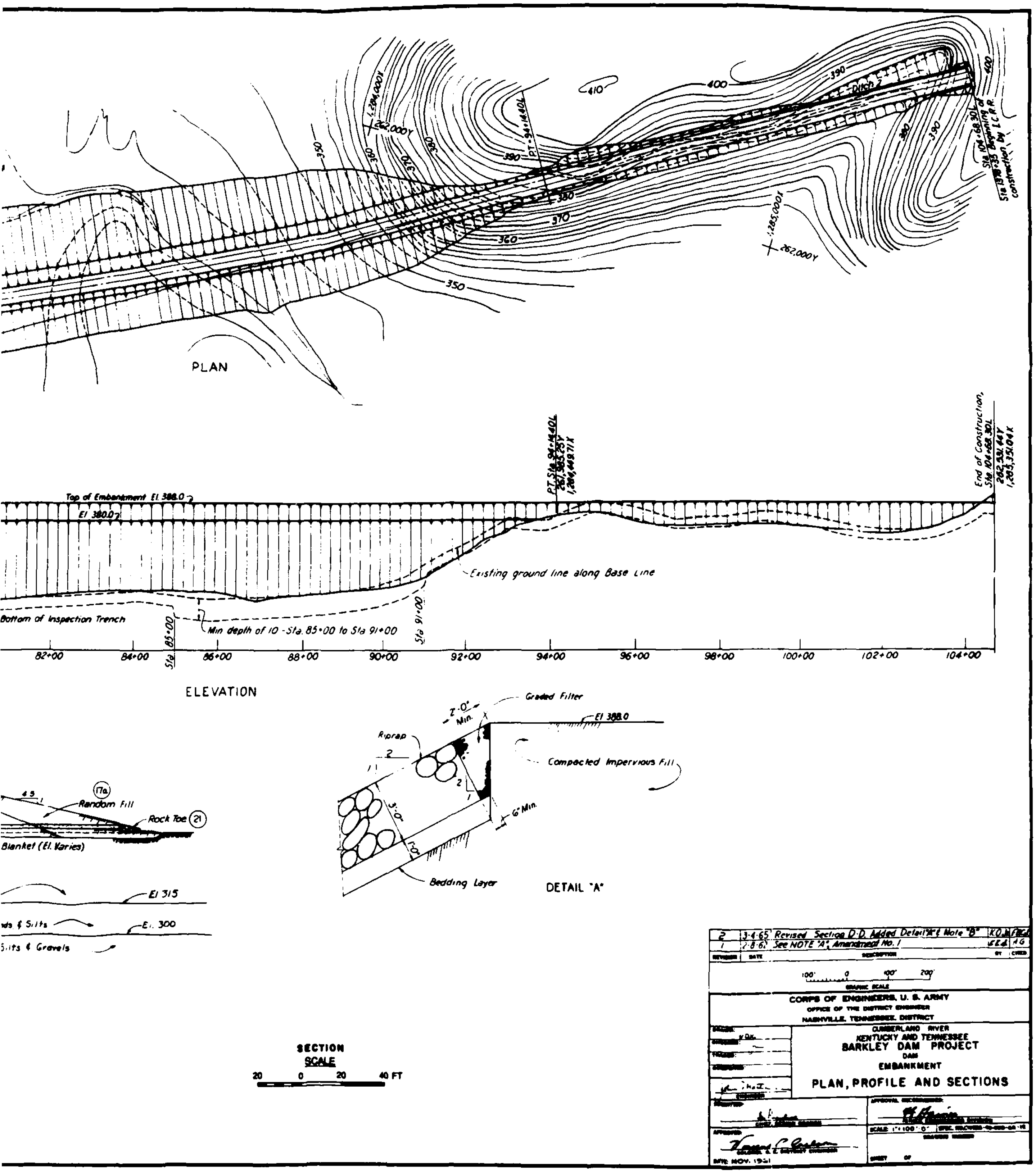


Figure 3. Dam embankment plan, profile and sectio

1 of 2



ure 3. Dam embankment plan, profile and sections

2 of 2

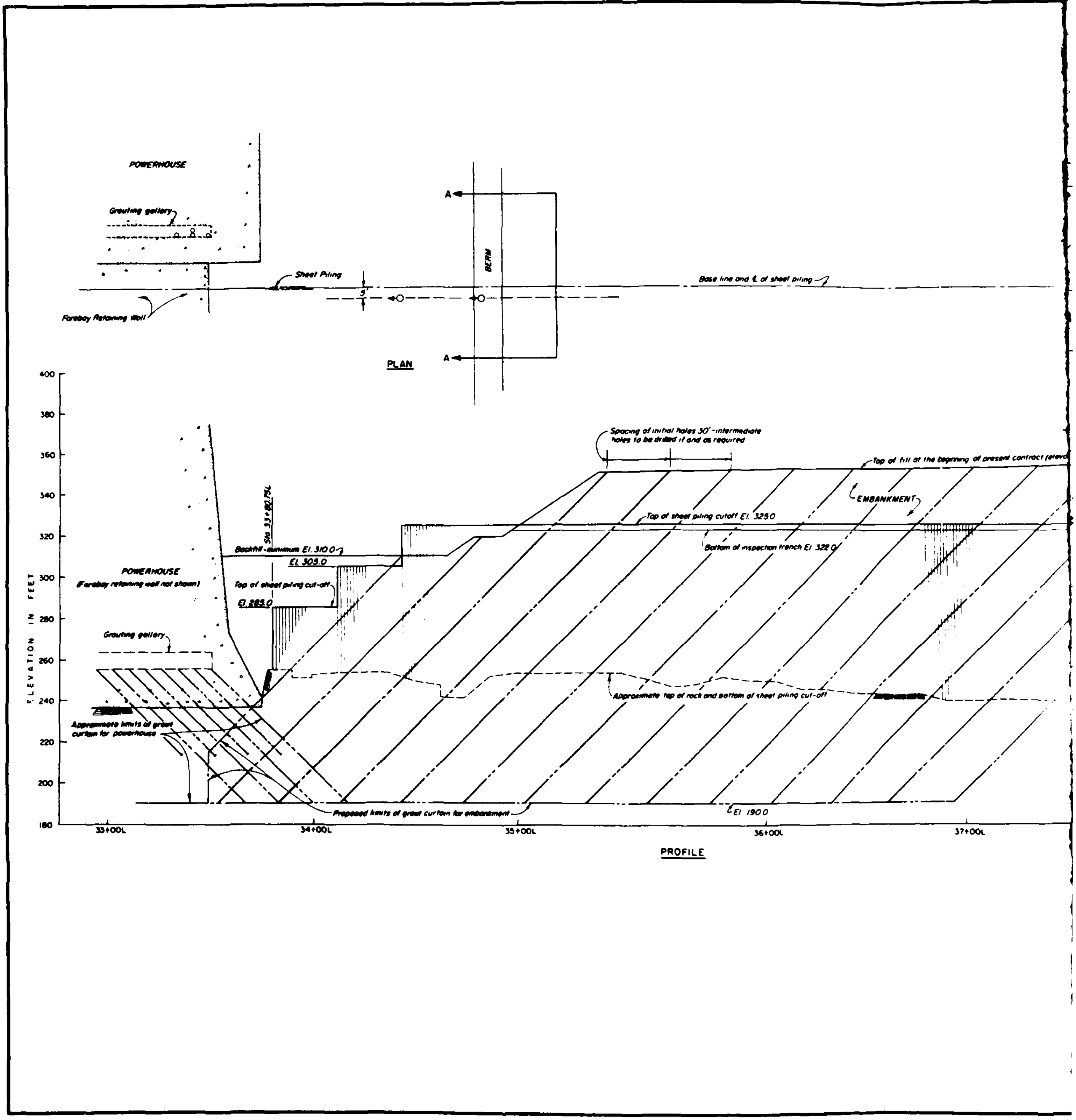
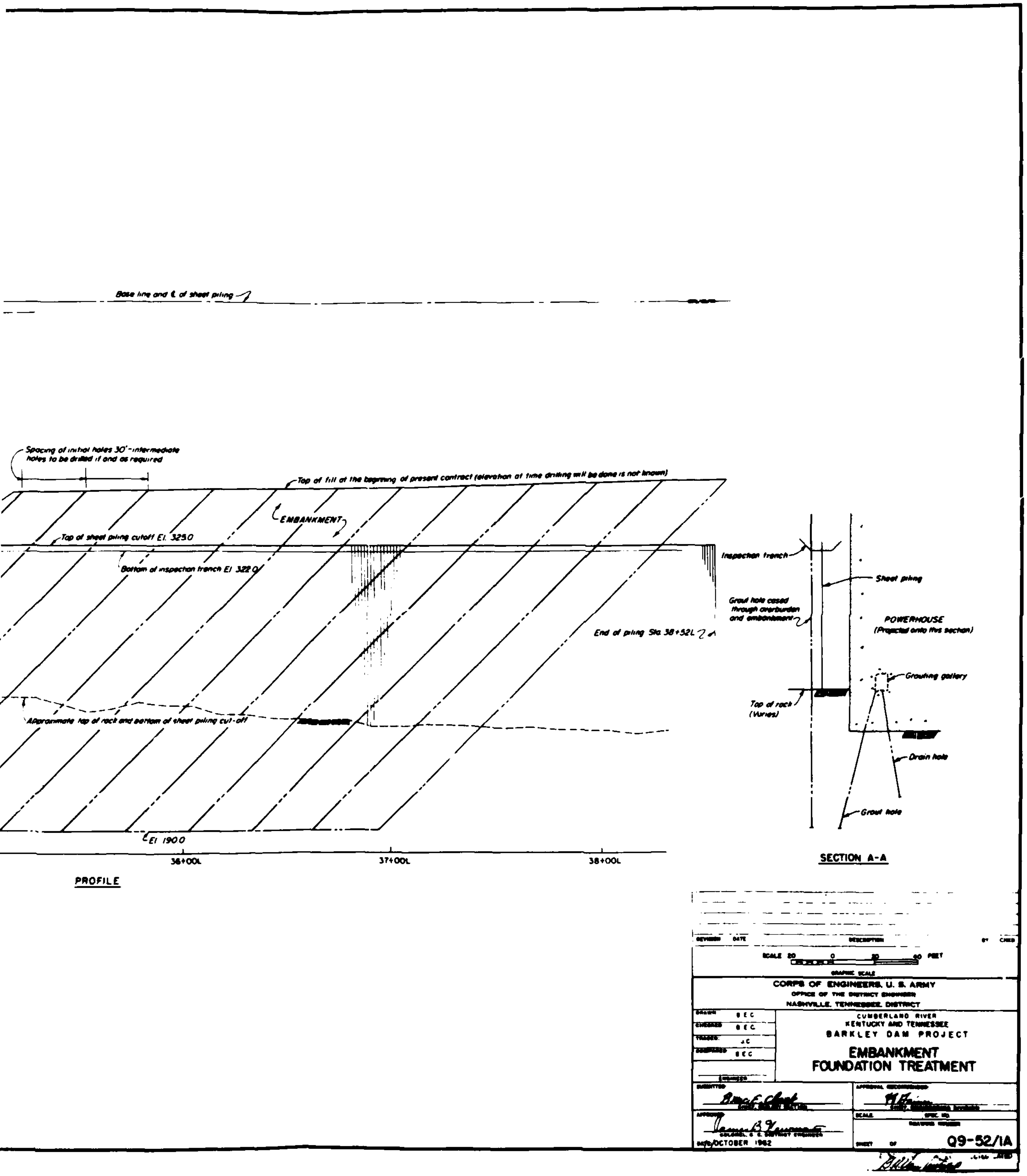


Figure 4. Embankment foundation treatment

1 of 2



igure 4. Embankment foundation treatment

2 of 2

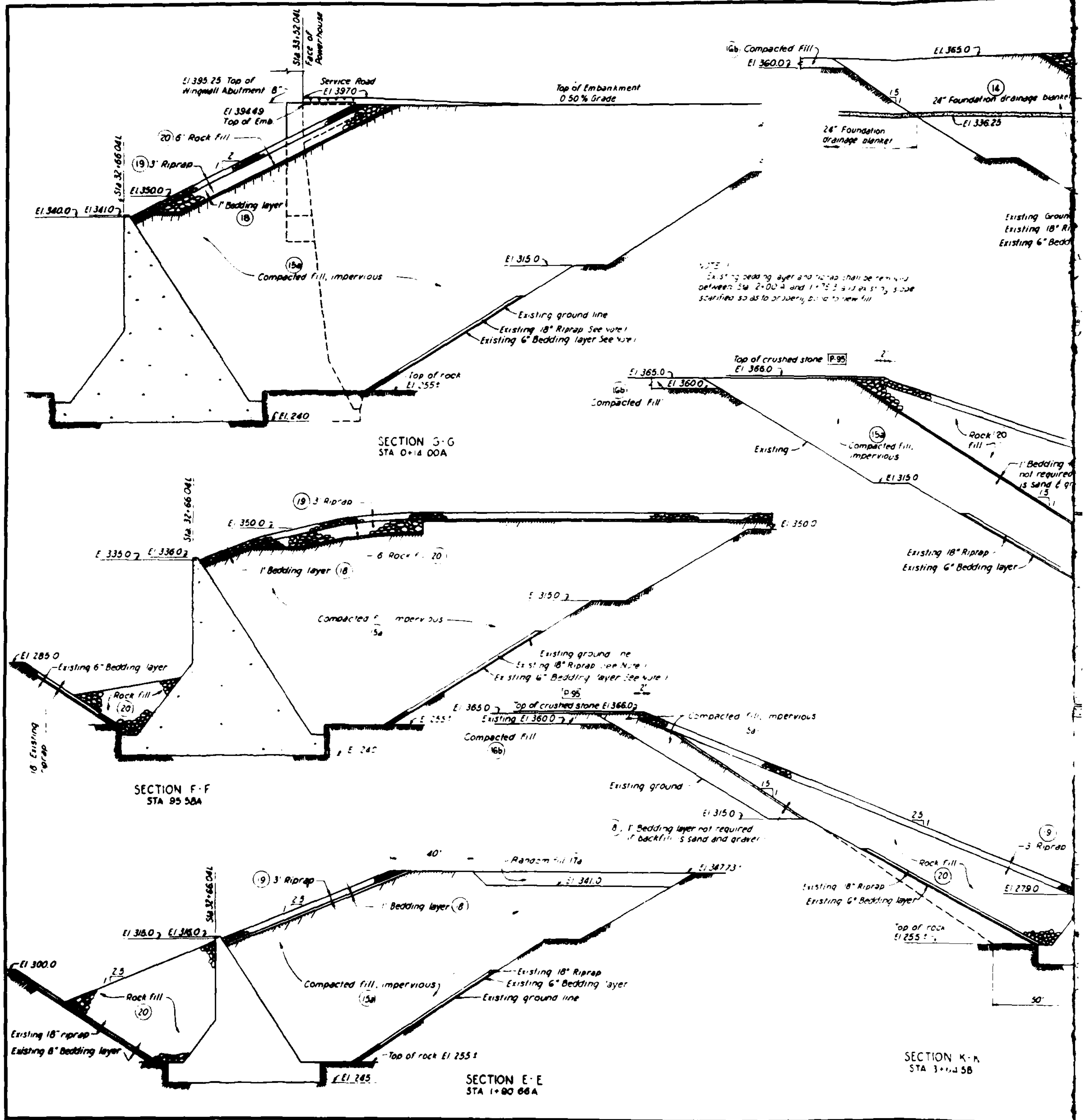


Figure 5. Dam embankment sections

1 of 2

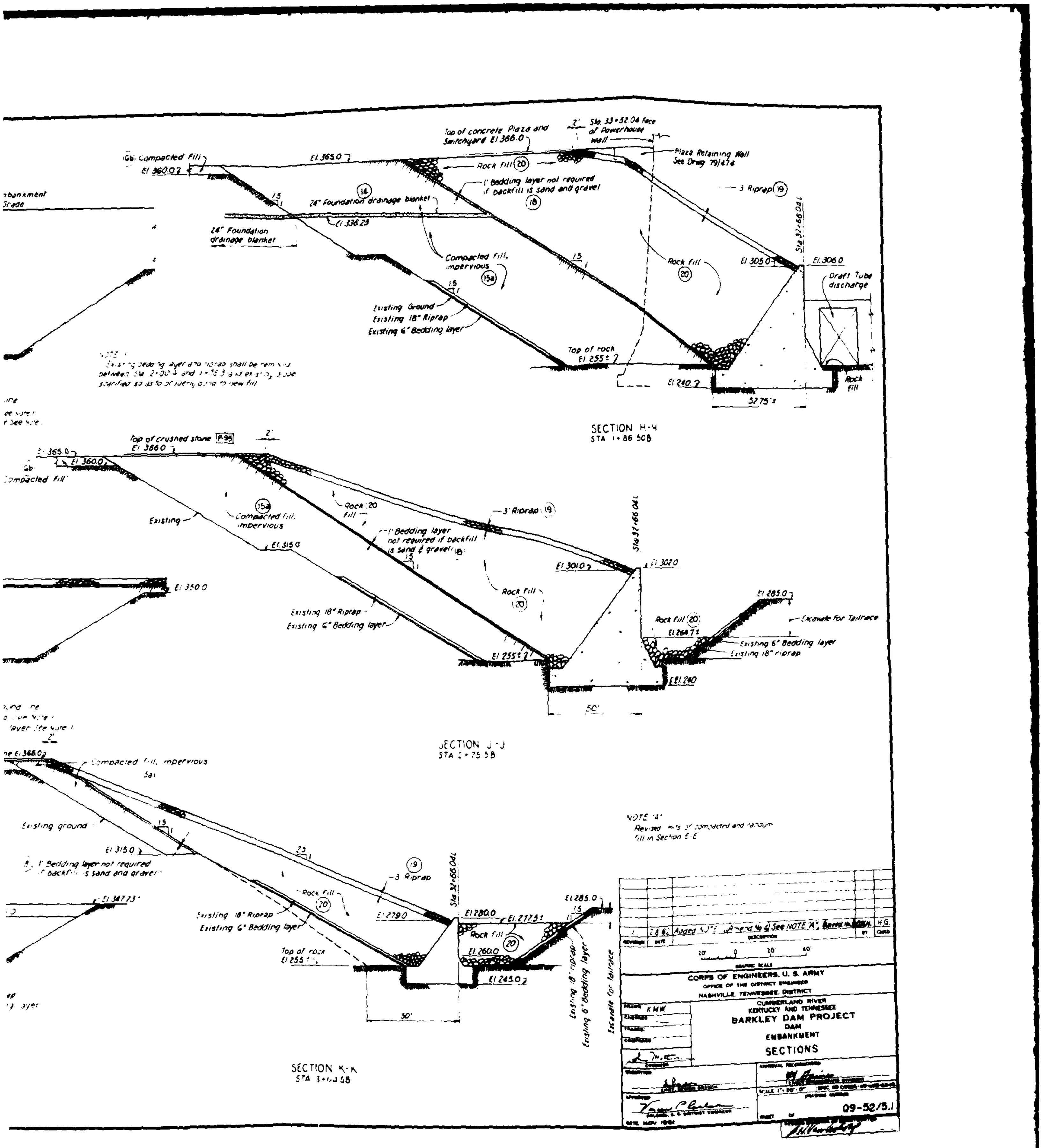


Figure 5. Dam embankment sections

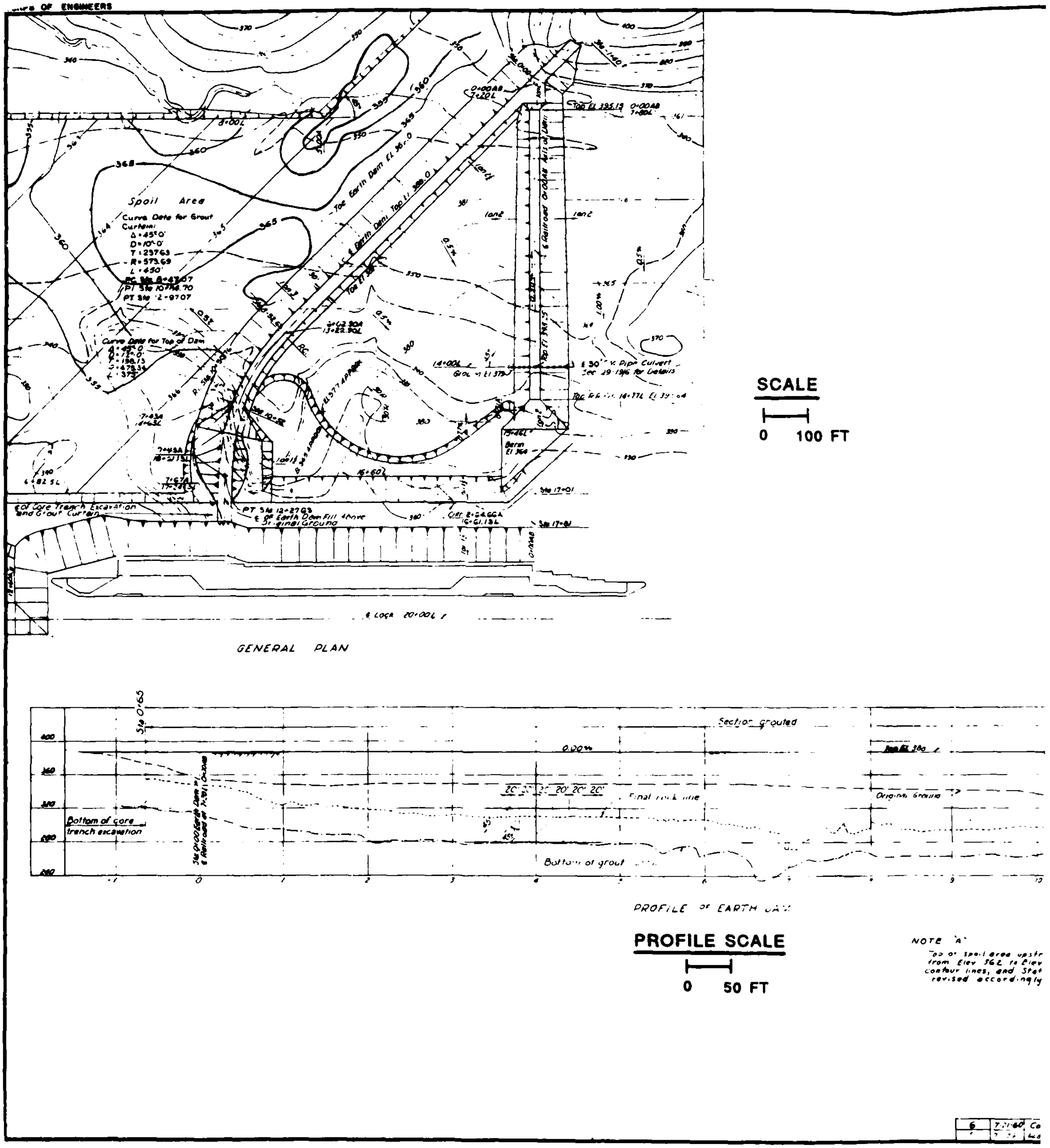


Figure 6. Earth dam plan and profile

1 of 2

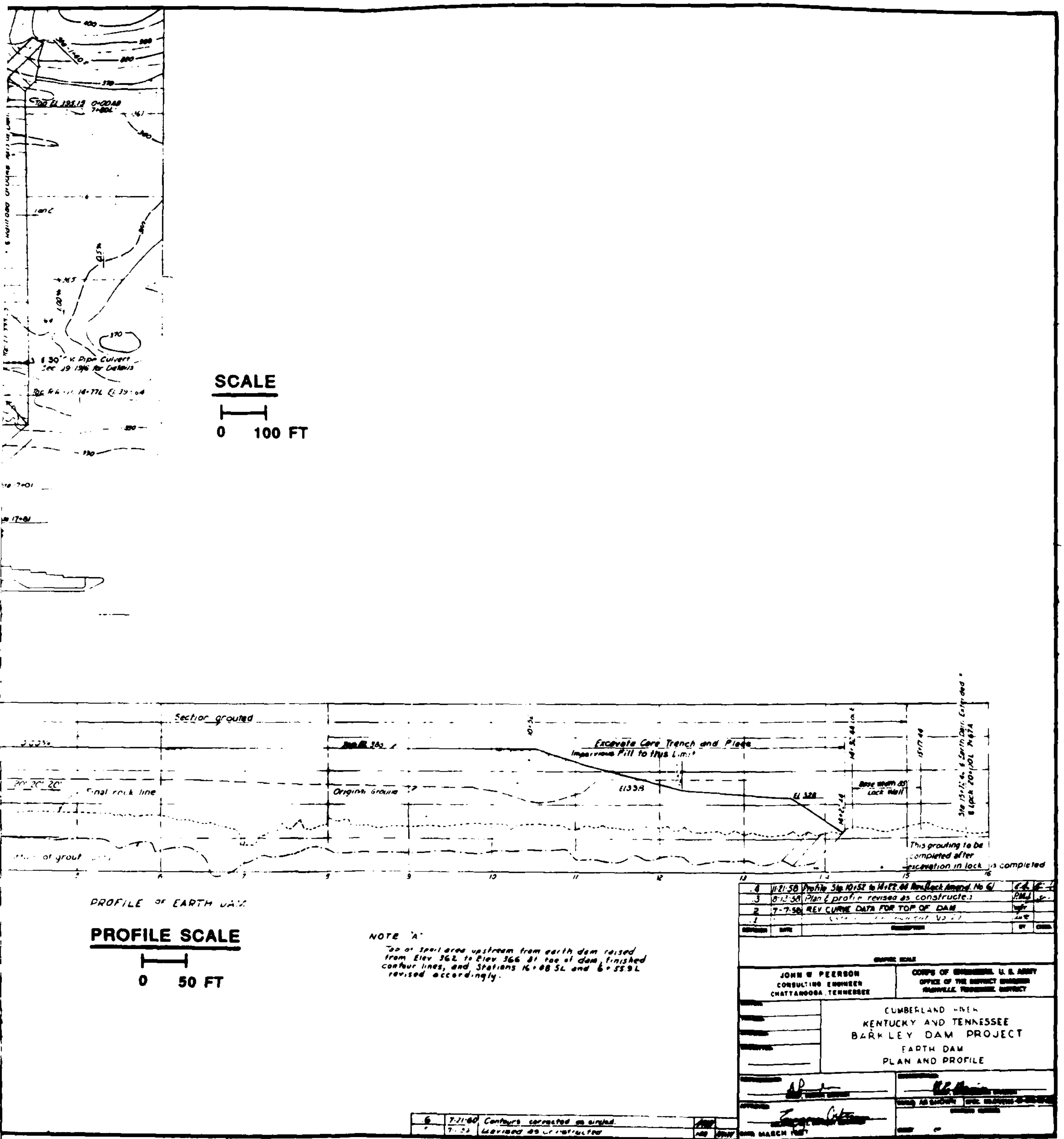


Figure 6. Earth dam plan and profile

2of

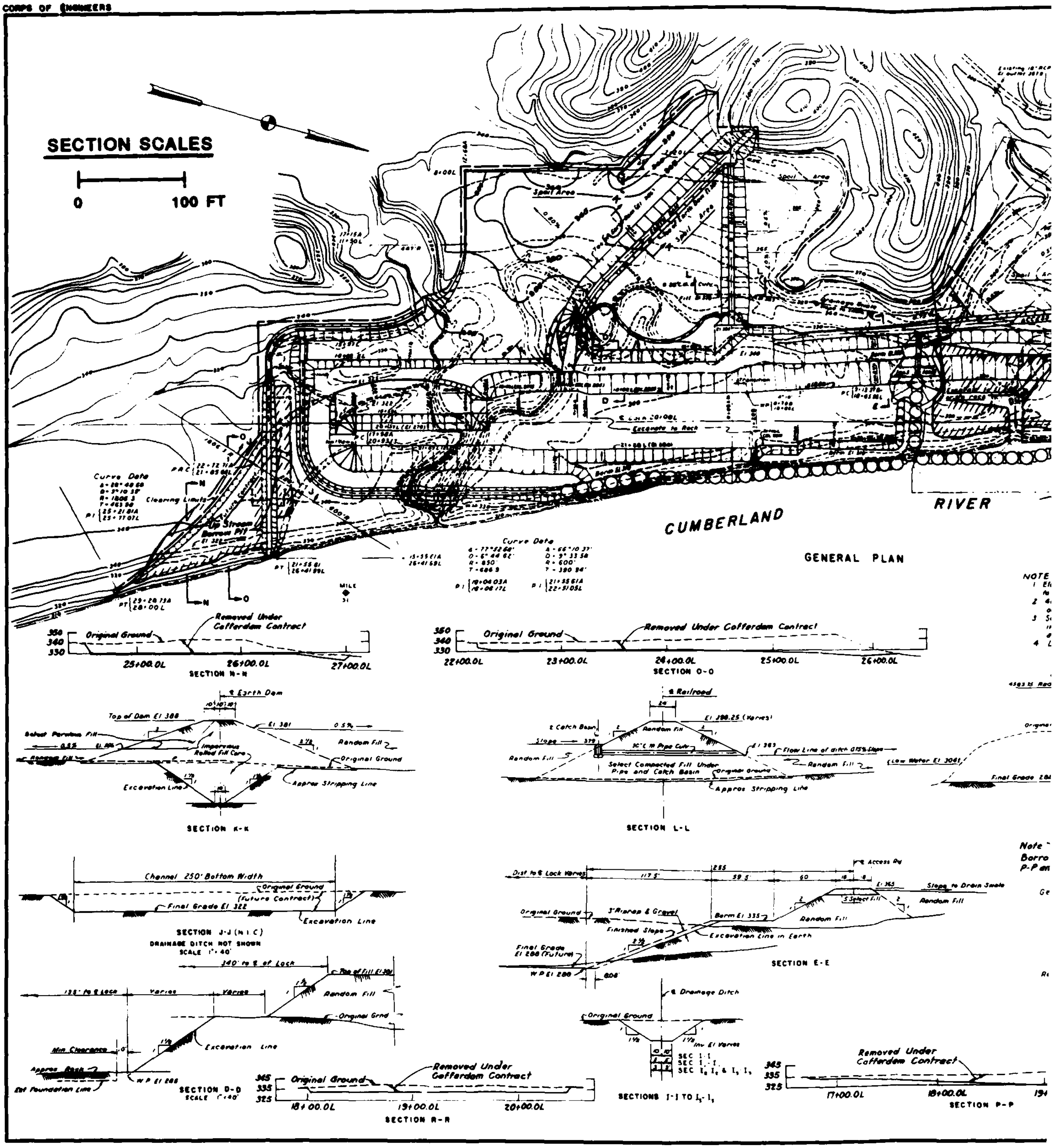
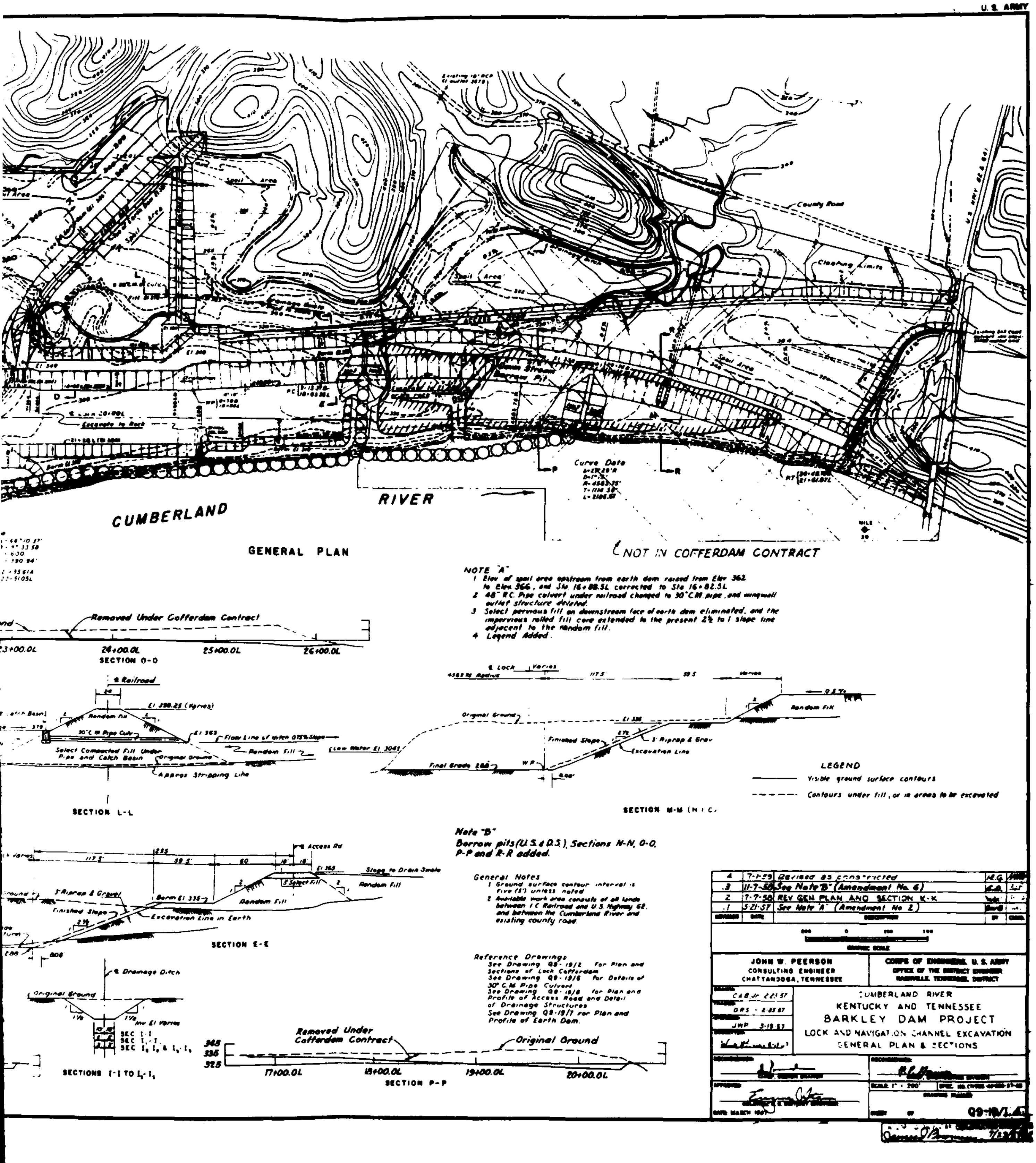


Figure 7. Lock and navigation channel excavation general |

1 of 2



ock and navigation channel excavation general plan and sections

2 of 2

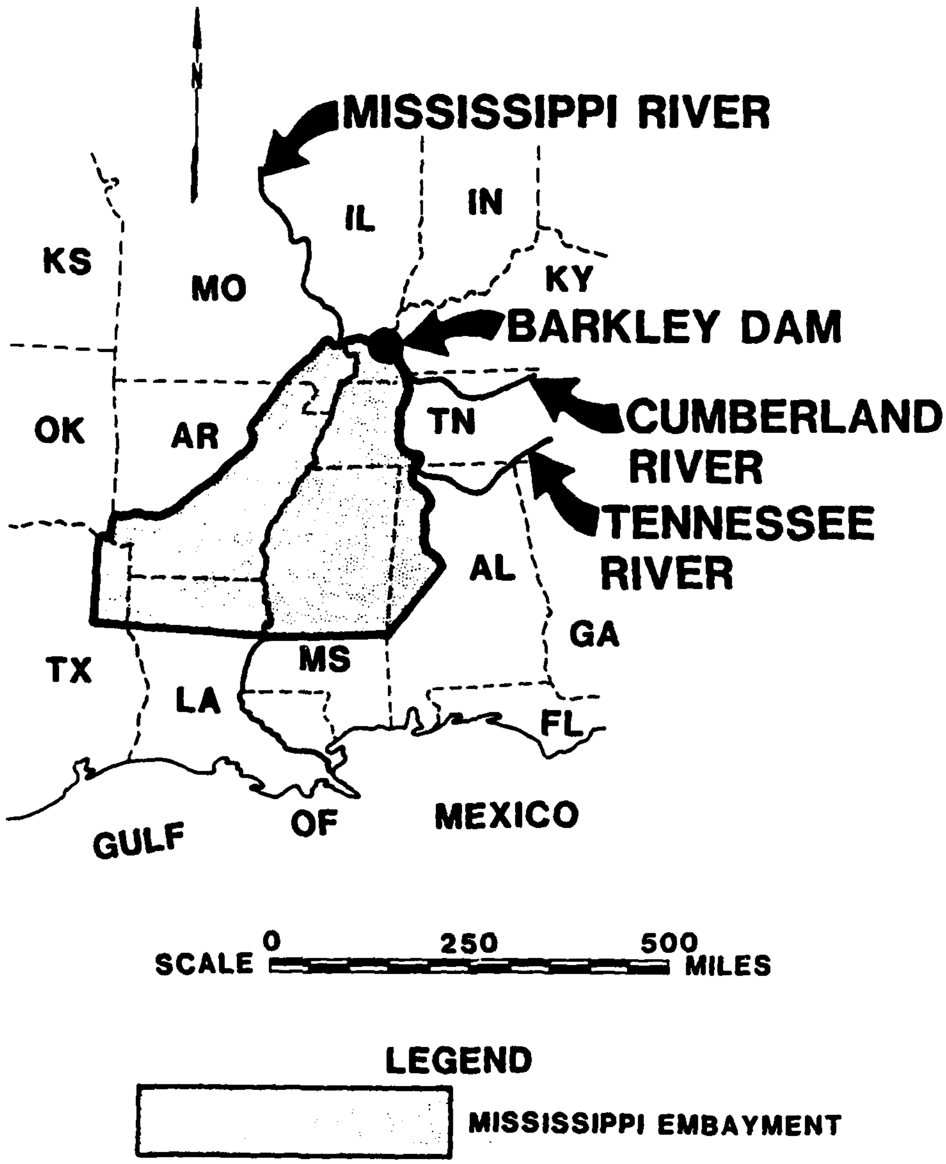


Figure 8. Map of the Mississippi embayment

ERA	PERIOD/SYSTEM (TIME) (ROCK)	EPOCH/SERIES (TIME) (ROCK)	GROUP	FORMATION	ROCK TYPE	THICKNESS (FT)	AGE (MY)
CENOZOIC	QUATERNARY	RECENT		ALLUVIUM (1)	SILT, CLAY, SAND & GRAVEL	0-120 (3)	0.01
		PLEISTOCENE		LOESS/CONTINENTAL DEPOSITS	SILT/SAND	0-7	
				GRAVEL	UNCONSOLIDATED GRAVEL & SAND	0-60+	1.6
	TERTIARY	PLIOCENE?		GRAVEL	UNCONSOLIDATED GRAVEL & SAND		5.3
		MIOCENE					23.7
		OLIGOCENE					36.6
		EOCENE					57.8
		PALEOCENE					70
MESOZOIC	CRETACEOUS	LATE/UPPER	NAVARRO	McNAIRY	UNCONSOLIDATED SAND & WEAKLY CONSOLIDATED GRAVEL	0-40+	
			WOODBINE	TUSCALOOSA	UNCONSOLIDATED SAND & WEAKLY CONSOLIDATED GRAVEL	0-110+	135
	JURASSIC						208
	TRIASSIC						245
PALEOZOIC	PERMIAN						286
	PENNSYLVANIAN						315
	MISSISSIPPIAN	MERAMECIAN		UPPER ST LOUIS	LIMESTONE	170+	
				LOWER ST LOUIS/SALEM	LIMESTONE	340-370	
				WARSAW (2)	LIMESTONE	200± (35±) (3)	
		OSAGE		FT. PAYNE (2)	CHERTY LIMESTONE	600±	350
	DEVONIAN	LATE/UPPER		CHATTANOOGA	SHALE	150±	
		EARLY/LOWER		CLEAR CREEK/BAILEY	CHERTY LIMESTONE	500±	400
	SILURIAN	LATE/UPPER	BASS ISLANDS	DECATUR	LIMESTONE	300	
		MIDDLE	BROWNSPORT	BROWNSPORT	LIMESTONE & SHALY LIMESTONE		
				LOUISVILLE	LIMESTONE		
			WAYNE	WALDRON	SHALE		
				LAUREL	LIMESTONE		
				OSGOOD	LIMESTONE		
		EARLY/LOWER		BRASSFIELD	CHERTY LIMESTONE		440
	ORDOVICIAN	CINCINNATIAN		MAQUOKETA	SHALE	300±	
		CHAMPLAINIAN		KIMMSWICK	LIMESTONE	4200	
				PLATTIN	LIMESTONE		
				JOACHIM	LIMESTONE		
				DUTCHTOWN	LIMESTONE		
		CANADIAN		UPPER KNOX	DOLOMITE		500
	CAMBRIAN	ST CROIXIAN		LOWER KNOX	DOLOMITE	1300±	
				BONTERRE	DOLOMITE & HARD SHALE	1500±	
				LAMOTTE & OLDER SEDIMENTS	DOLOMITIC & LIMY HARD SANDSTONE & SILTSTONE	3000±	600
	PRECAMBRIAN			—	HARD IGNEOUS & METAMORPHIC ROCKS		

(1) FOUNDATION FOR EMBANKMENT
(2) FOUNDATION FOR CONCRETE STRUCTURE
(3) THICKNESS AT DAM

Figure 9. Geologic column and time scale for the Barkley Lock and Dam

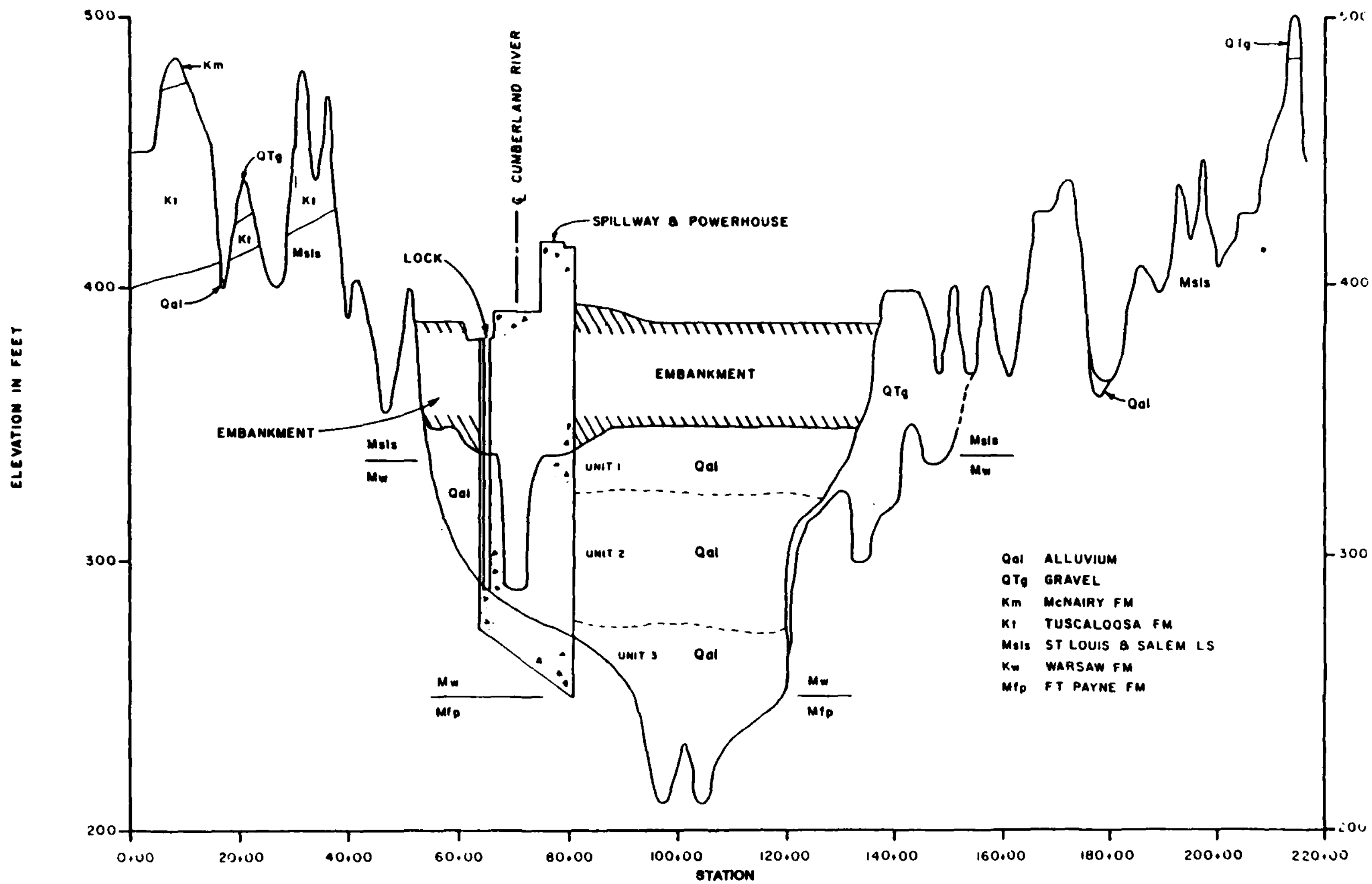
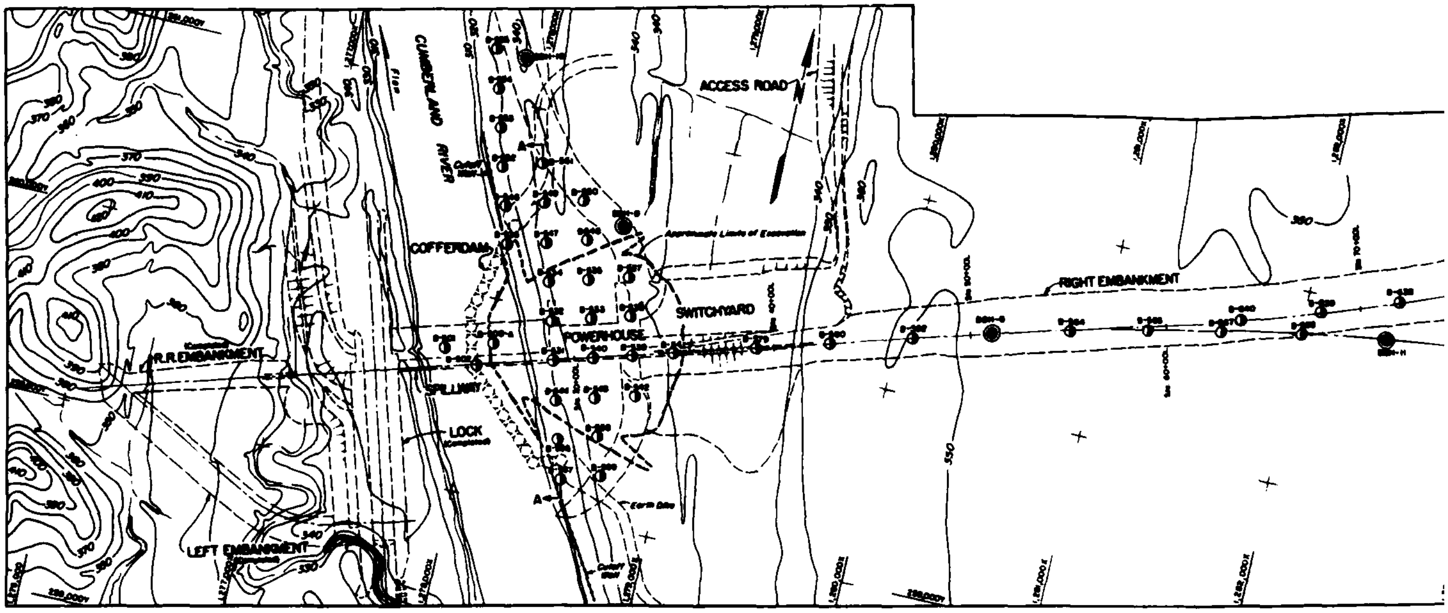
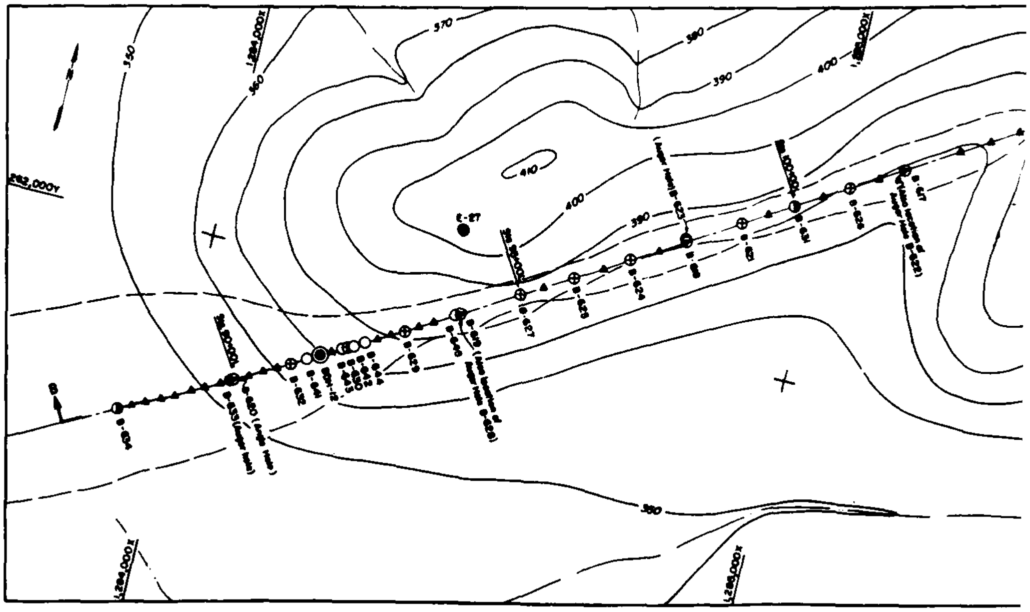


Figure 10. Generalized geologic section approximately along centerline of dam



GENERAL PLAN OF SOIL EXPLORATIONS

SCALE 300 0 300 600 FEET



DETAIL PLAN OF EAST ABUTMENT EXPLORATIONS

SCALE 100 0 100 200 FEET

Figure 11. Plan of soil explorations

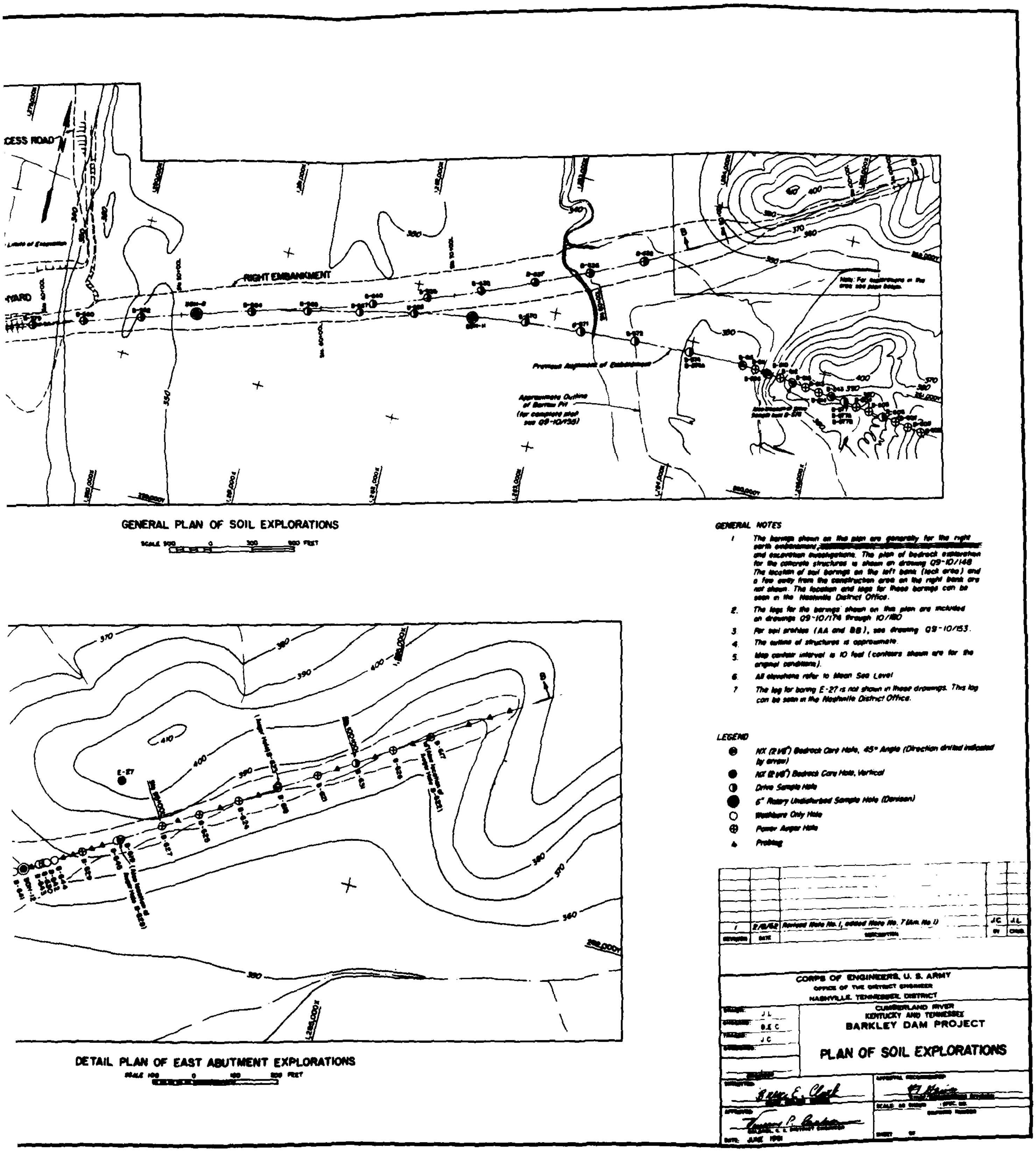


Figure 11. Plan of soil explorations

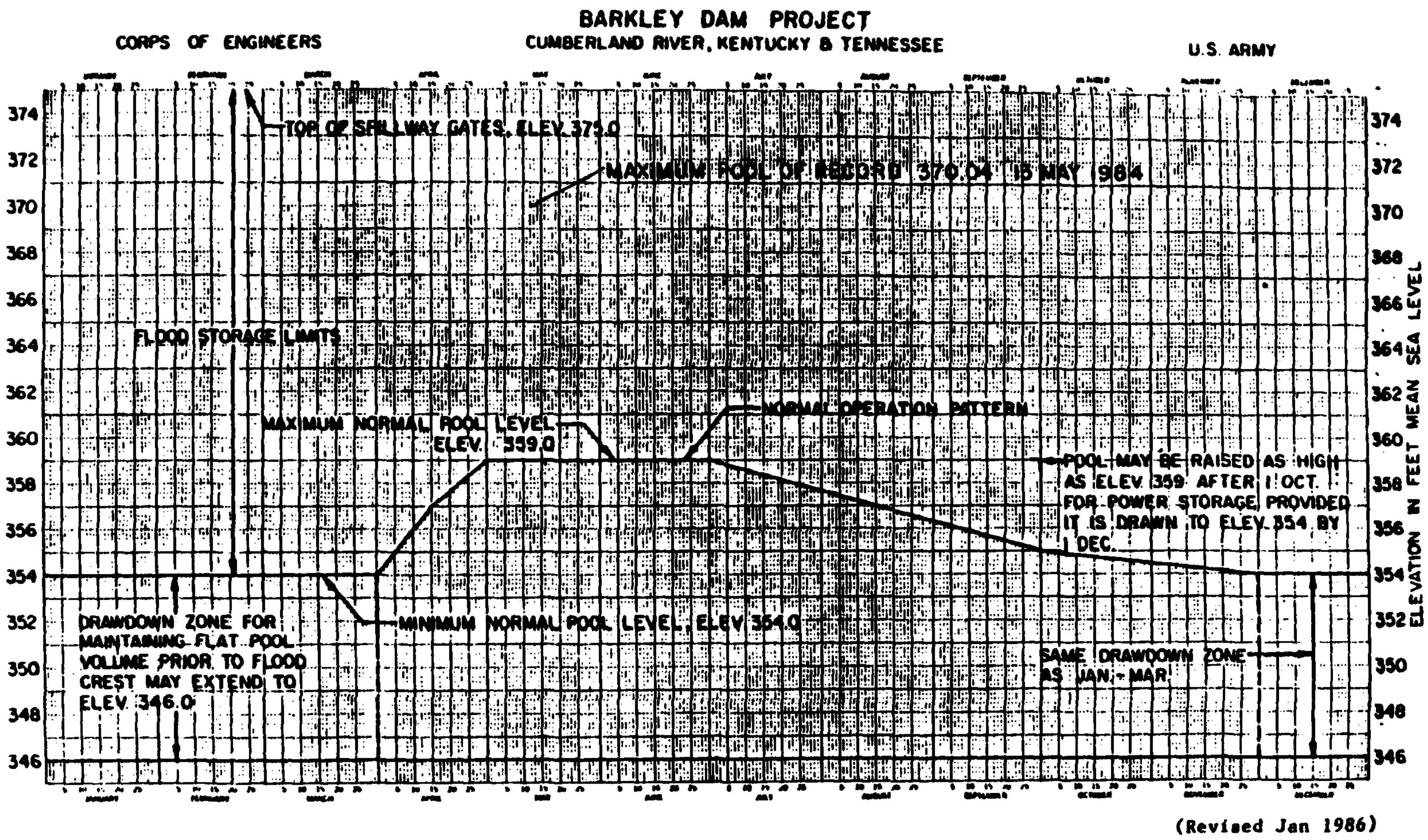
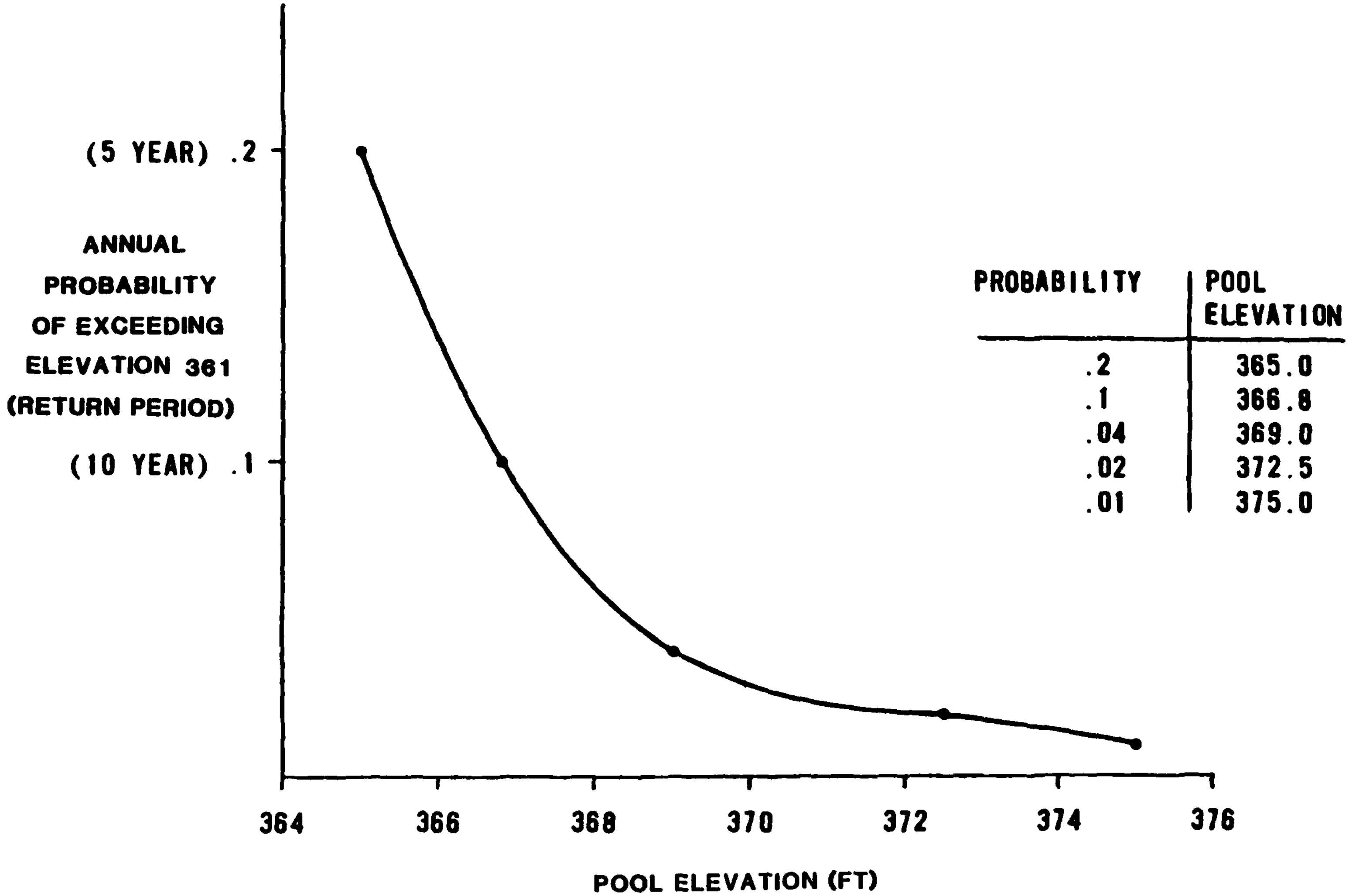


Figure 12. Guide curve for reservoir levels of Barkley Dam



PROBABILITY	POOL ELEVATION
.2	365.0
.1	366.8
.04	369.0
.02	372.5
.01	375.0

Figure 13. Annual probability of exceeding elevation 361 ft versus pool elevation

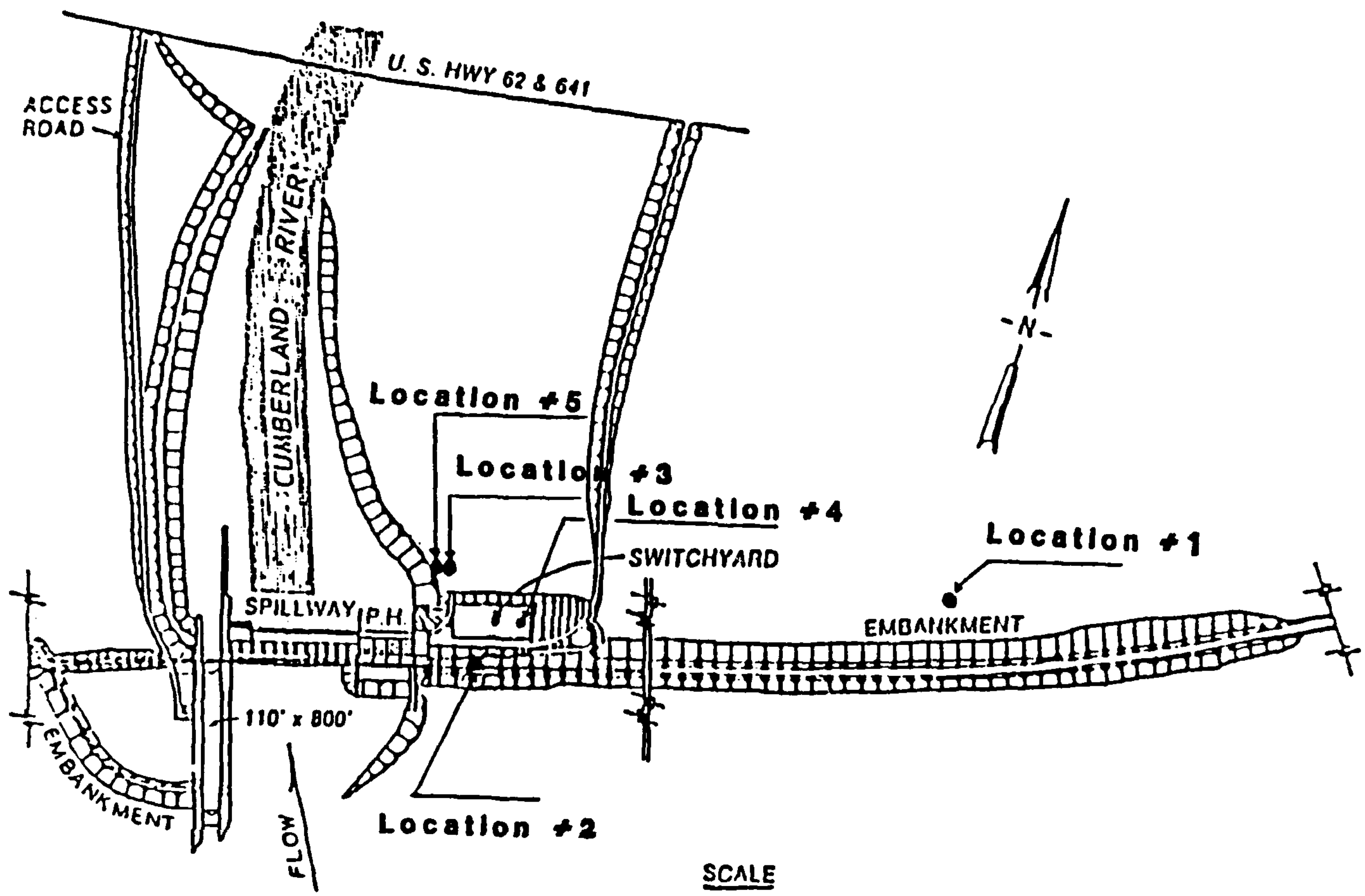


Figure 14. Plan of geophysical test locations at Barkley Dam

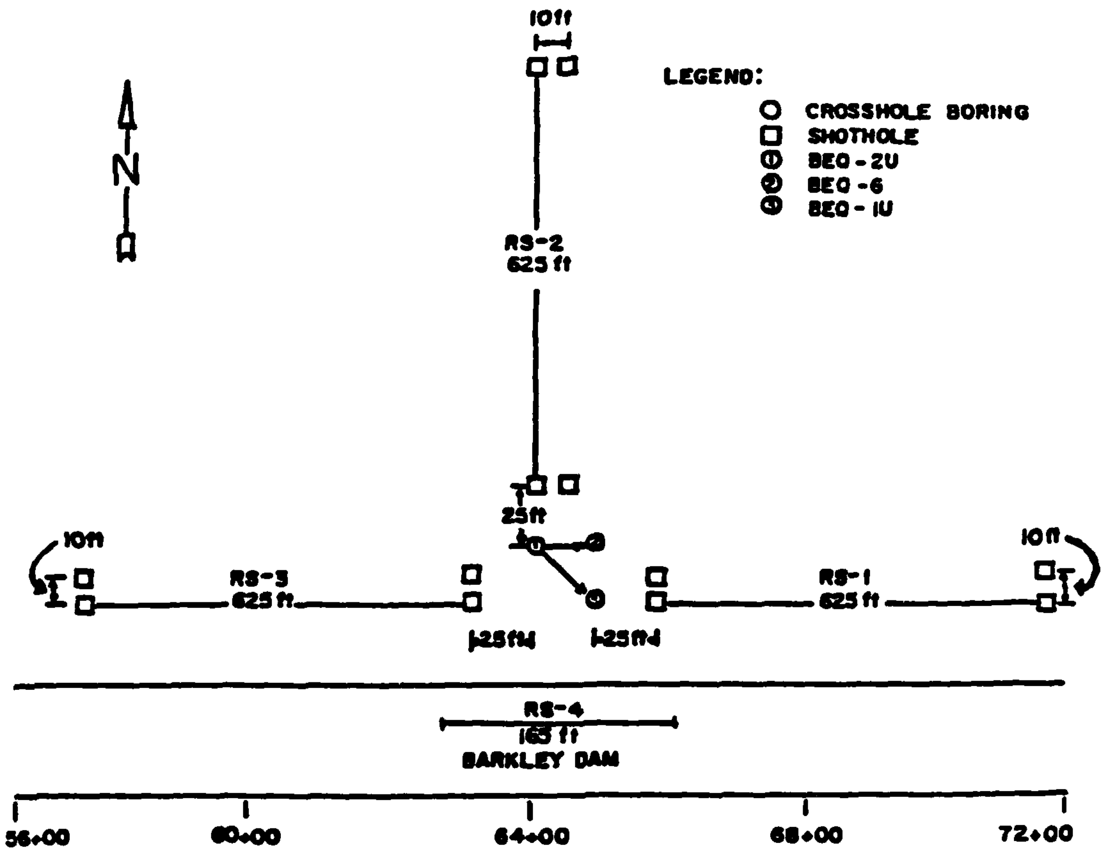


Figure 15. Layout of geophysical tests at Location 1

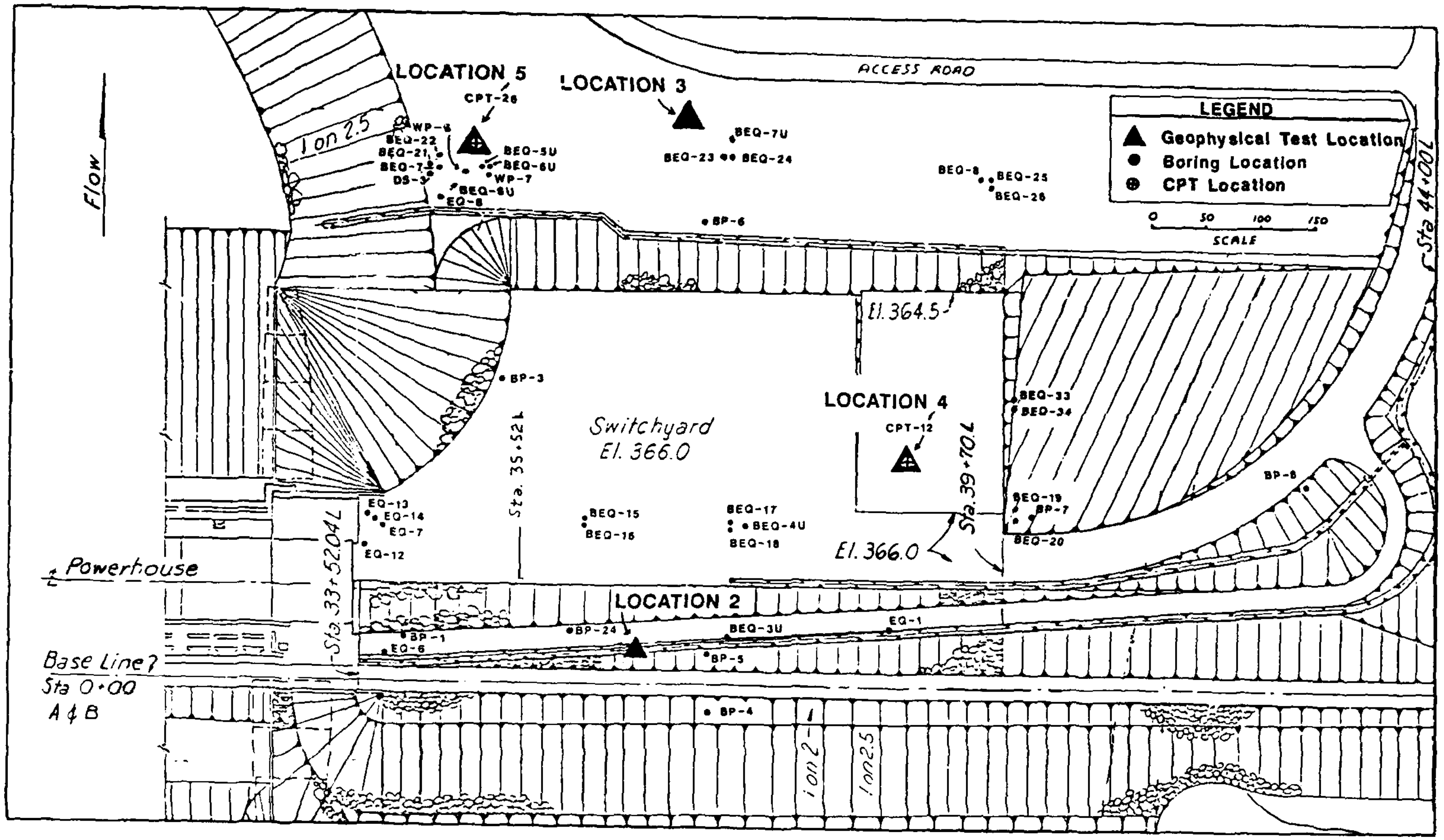


Figure 16. Detailed plan of field investigations in switchyard area at Barkley Dam

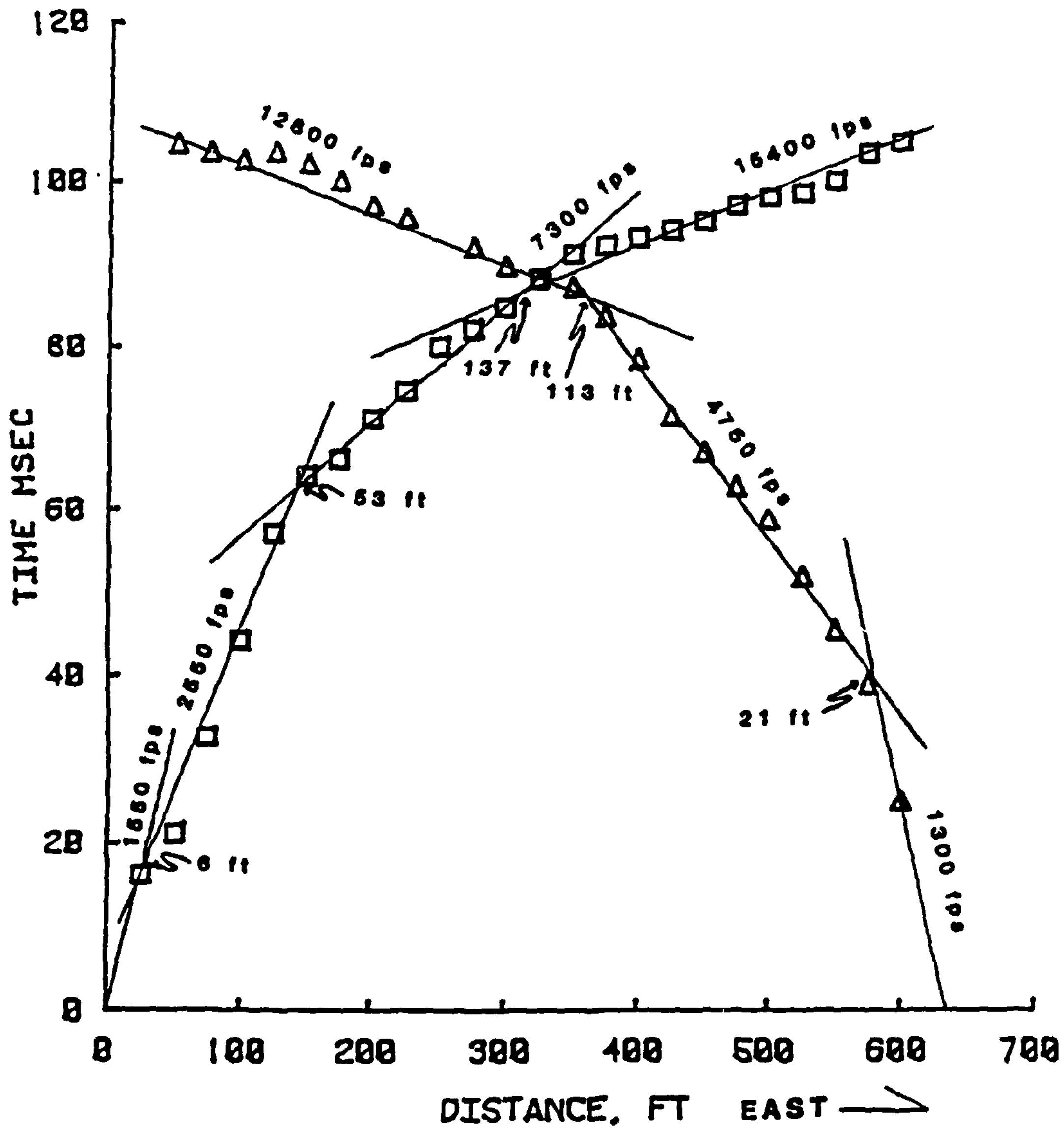


Figure 17. Refraction seismic survey, line RS-1-P

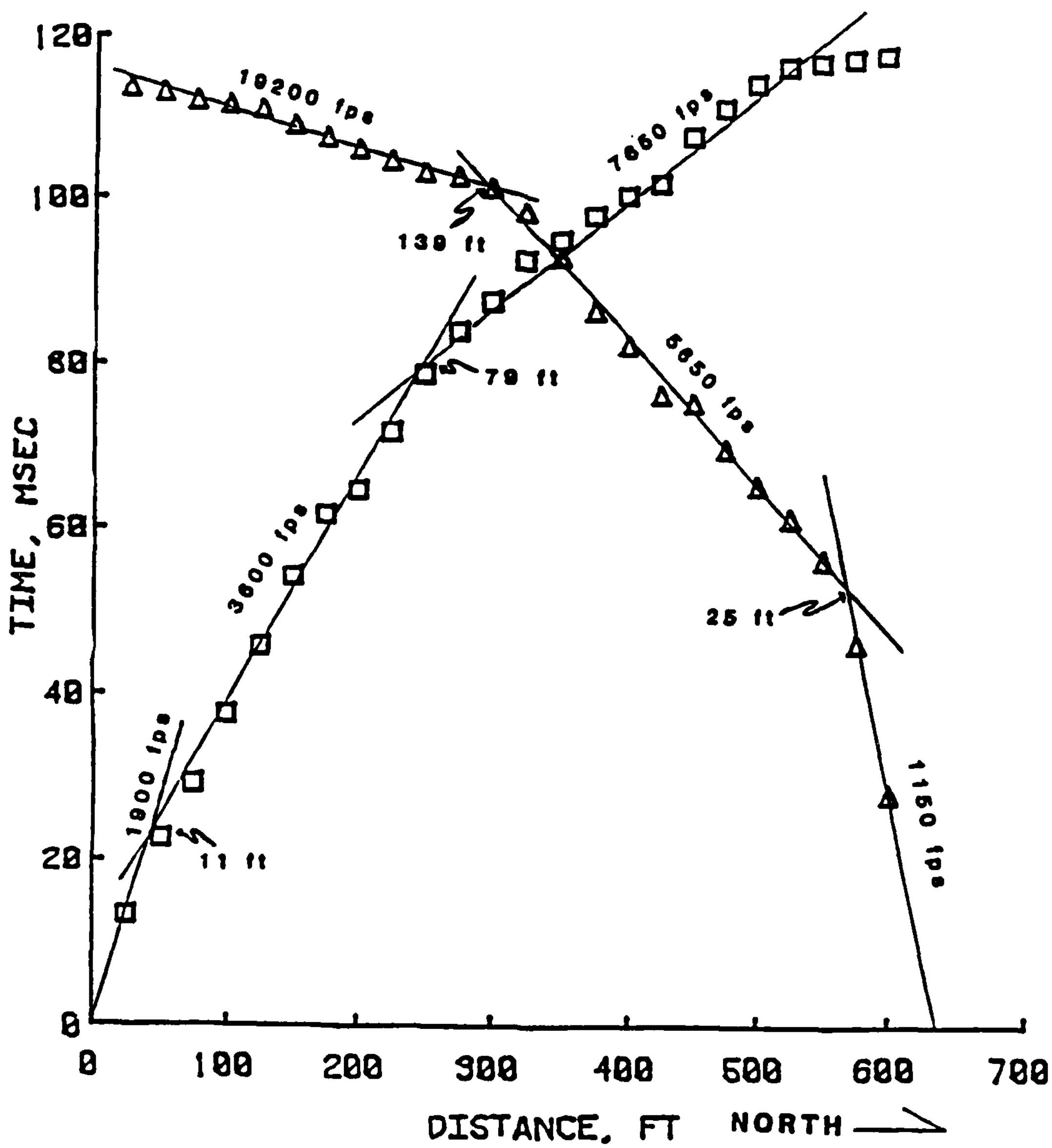


Figure 18. Refraction seismic survey, line RS-2-P

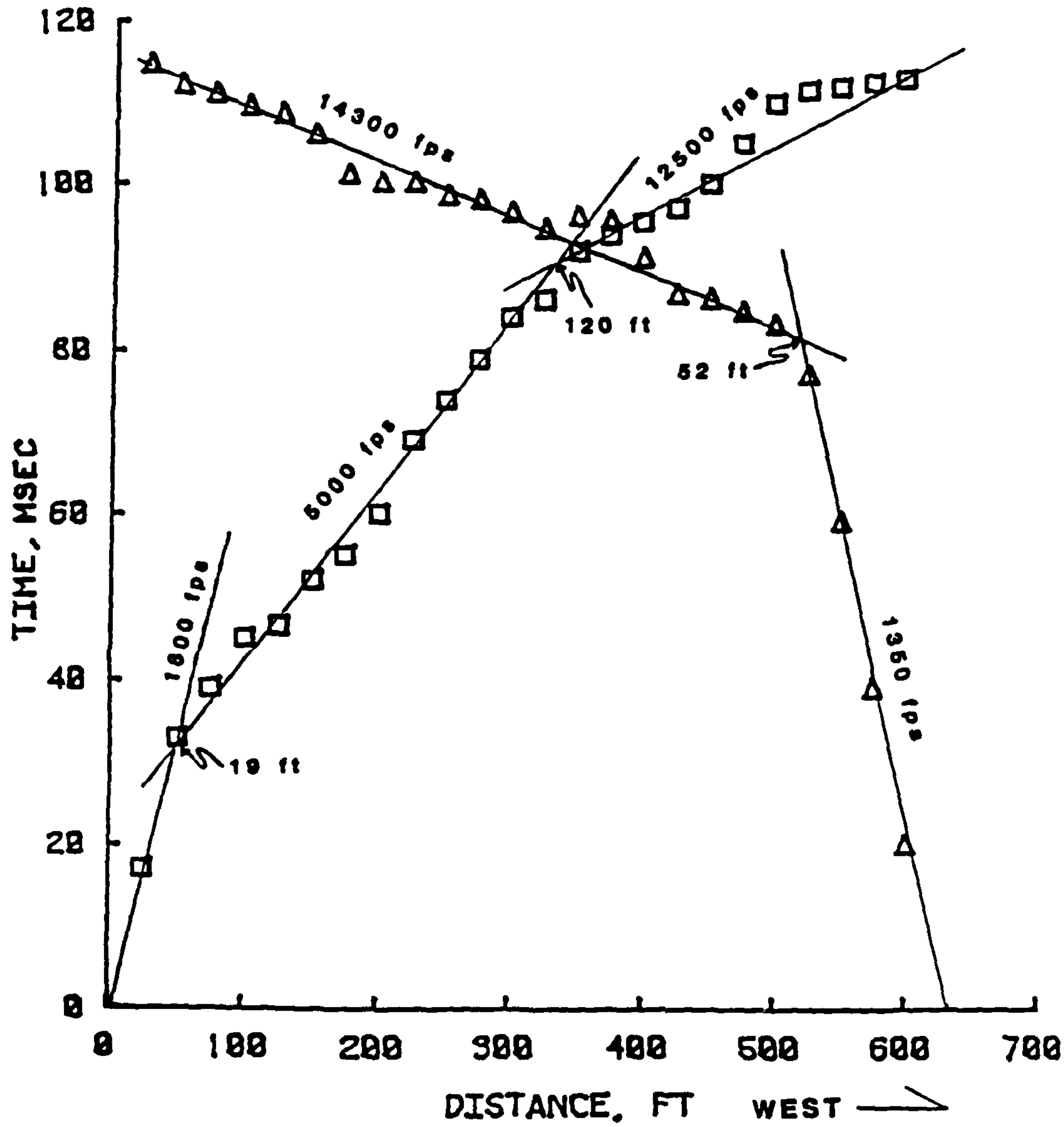


Figure 19. Refraction seismic survey, line RS-3-P

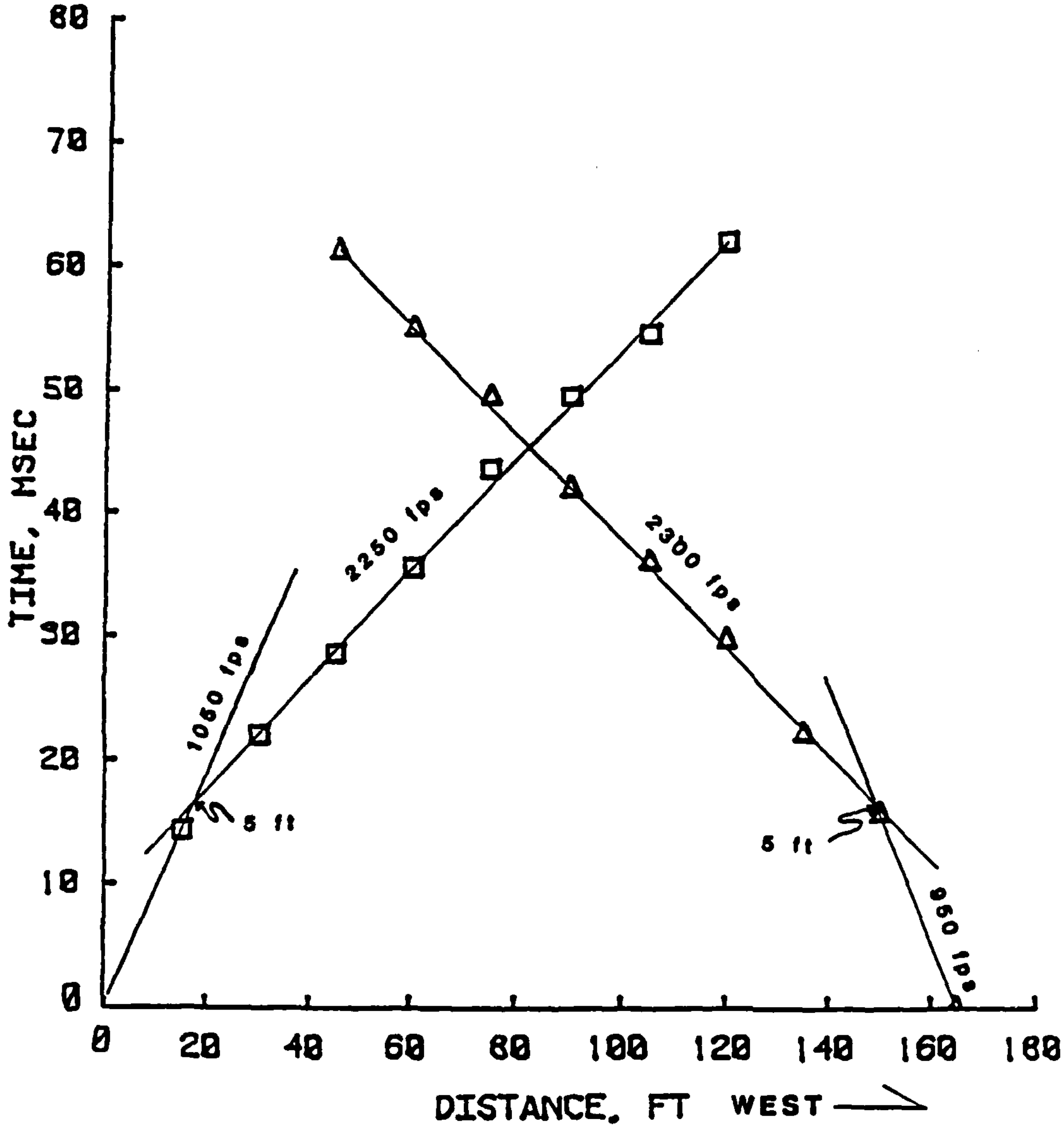


Figure 20. Refraction seismic survey, line RS-4-P

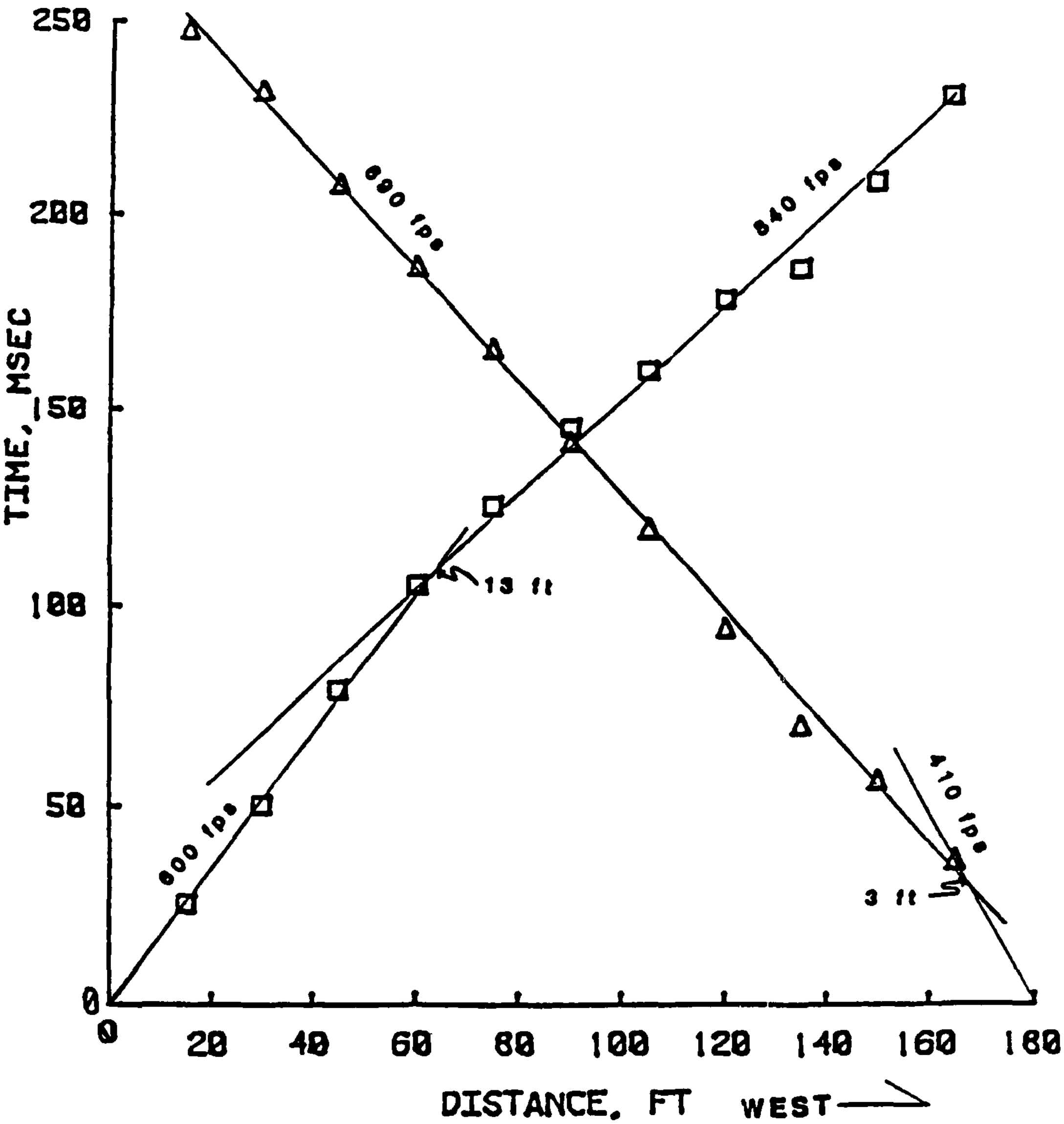


Figure 21. Refraction seismic survey, line RS-3-S

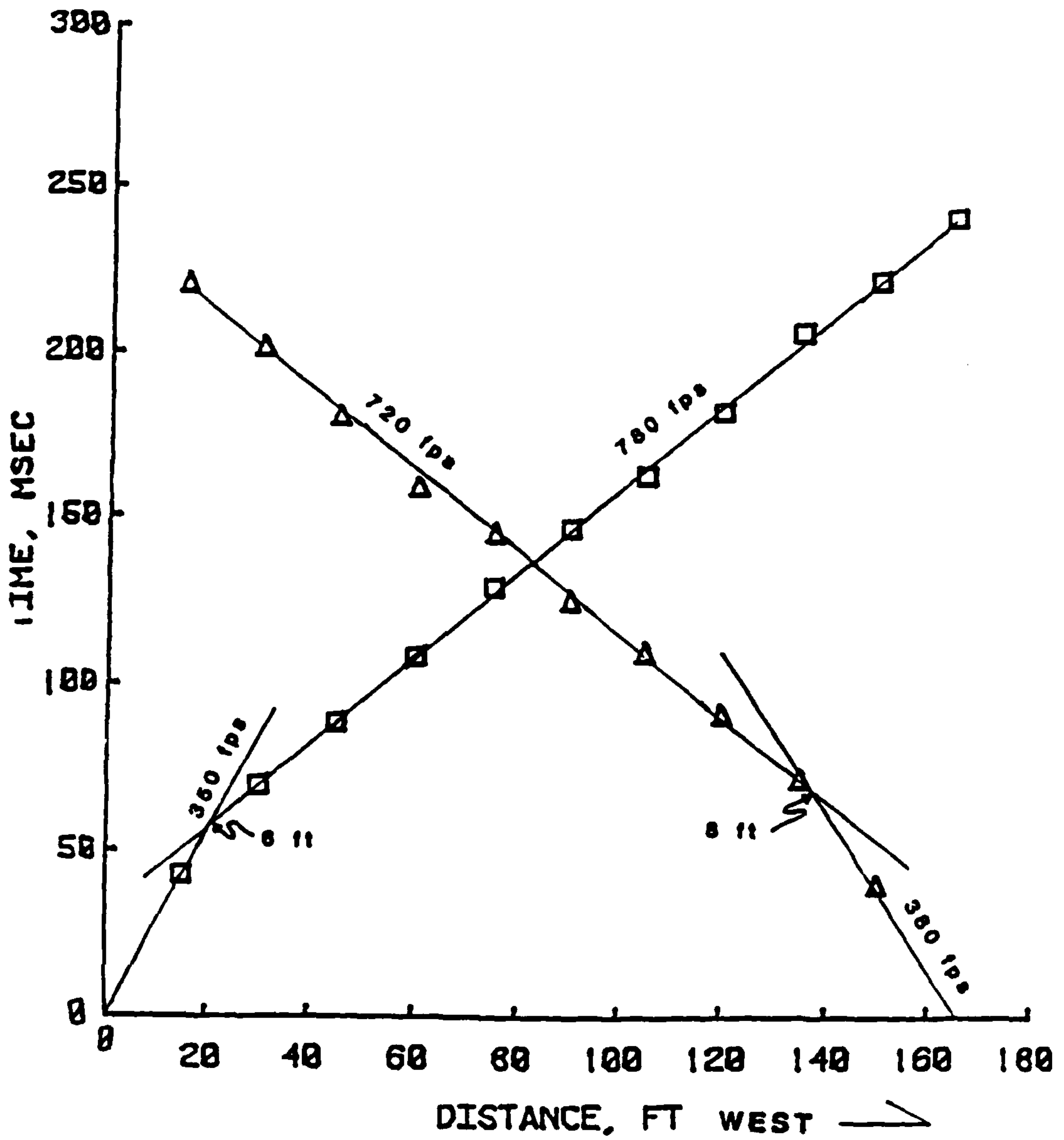


Figure 22. Refraction seismic survey, line RS-4-S

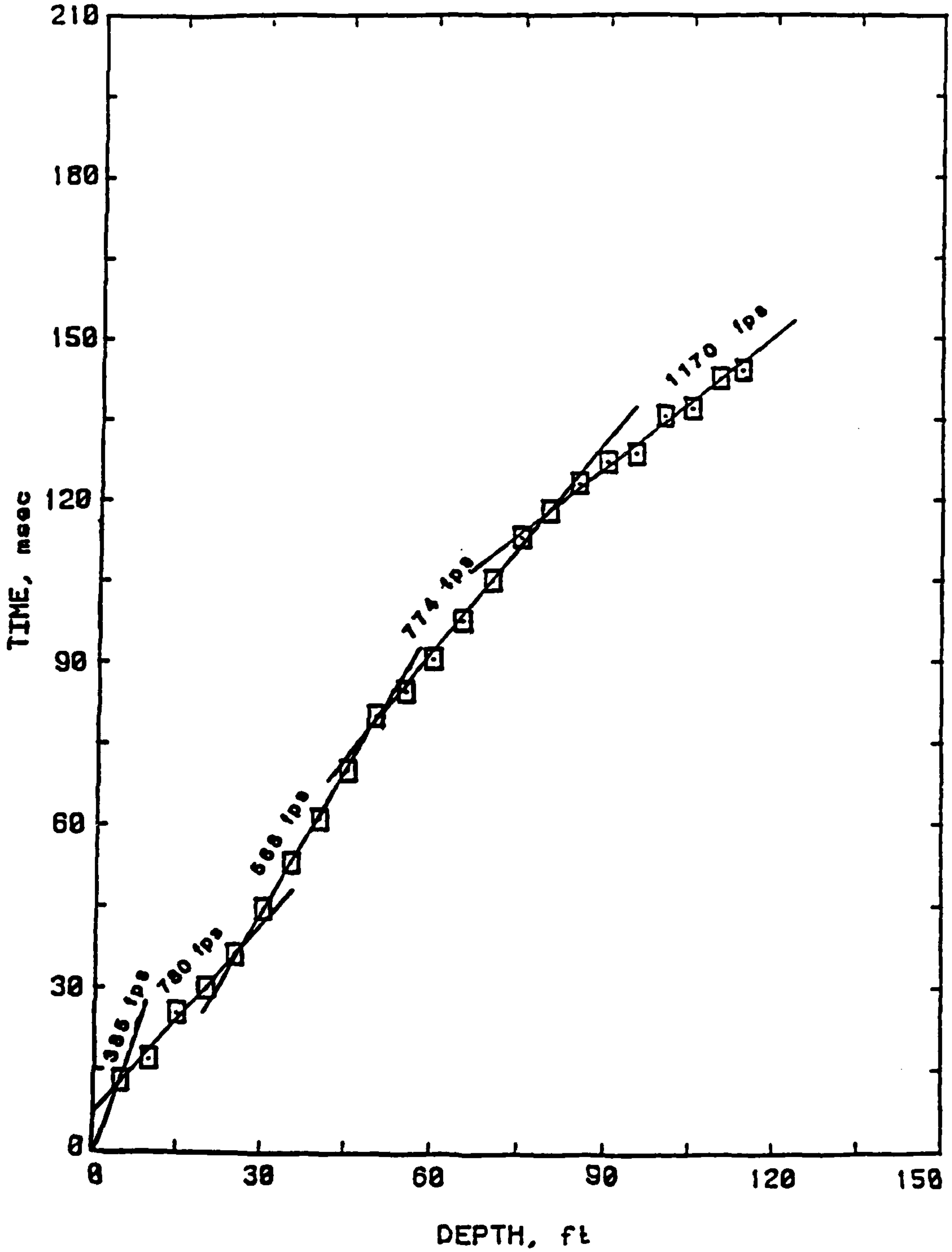


Figure 23. Downhole S-wave survey conducted in Boring BEQ-2U at Location 1

Depth (ft)	S-Wave Velocity (fps) 1 to 2	S-Wave Velocity (fps) 1 to 3	Downhole	P-Wave Velocity (fps) 1 to 2	P-Wave Velocity (fps) 1 to 3
	420	420	385		
5	753	693	780	1,776	1,656
10	753	771		1,573	1,656
15	753	771		1,784	1,656
20	648	673		1,784	1,656
25	648	673		5,883	6,327
30	510	673		5,883	6,327
35	510	637	568	4,594	5,190
40	483	589		4,594	5,190
45	483	589		6,935	7,132
50	545	662		6,935	7,132
55	545	746		5,963	5,687
60	545	746		5,963	5,687
65	545	746	774	6,955	6,291
70	673	746		6,955	6,291
75	673	746		5,963	6,291
80	952	1,110		5,963	6,291
85	952	1,110		8,274	9,304
90	880	1,026		6,913	6,948
95	880	1,026	1,170	6,913	6,948
100	1,021	1,026		8,359	7,936
105	1,021	1,026		6,980	6,939
110	—	—		6,980	7,924
	—	—			
115	—	—		14,089	13,888

Figure 24. Tabulation of CROSSHOLE program output of S-wave and and P-wave velocities from crosshole tests at Location 1. Downhole results shown for comparison

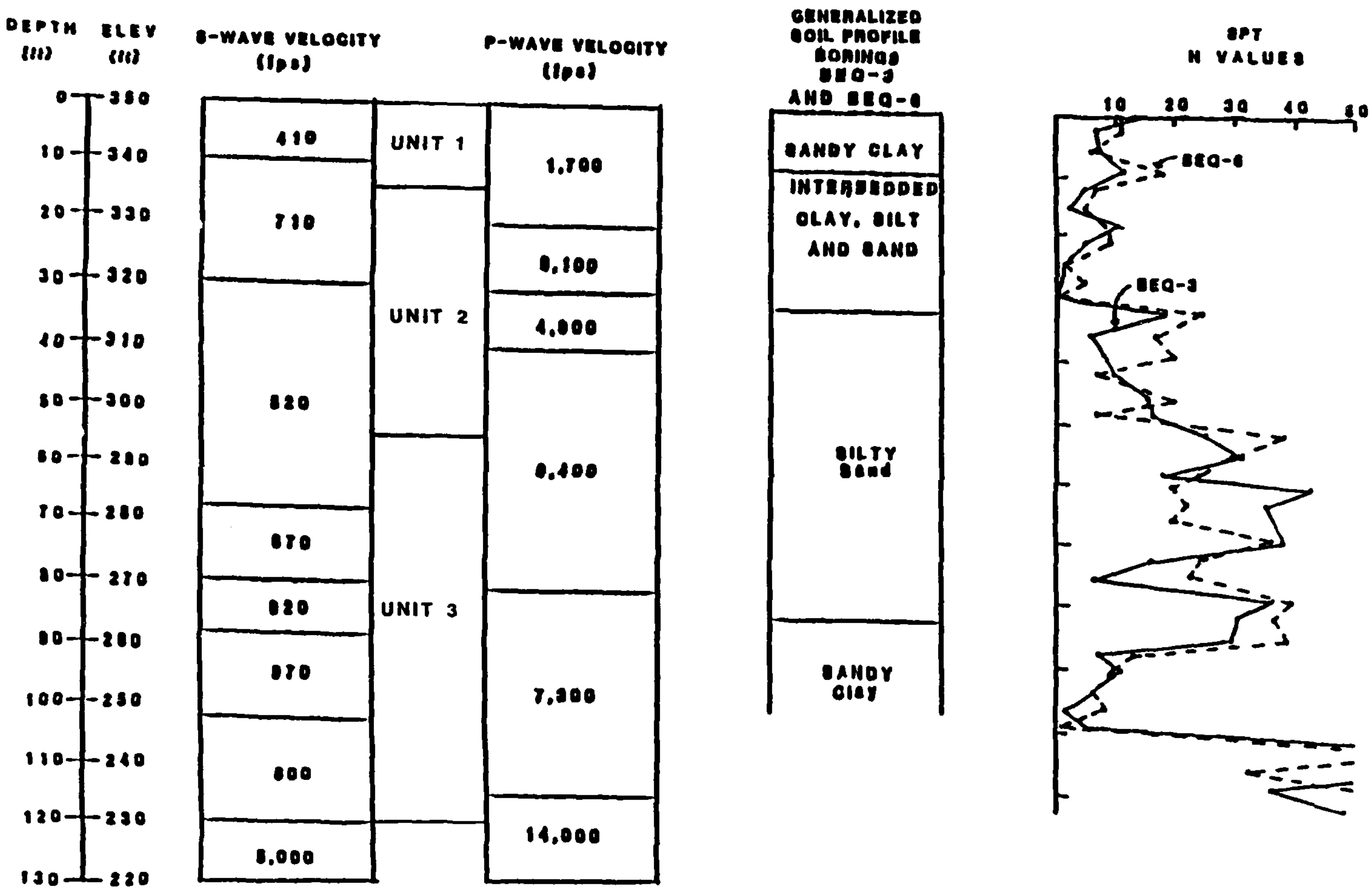


Figure 25. S-wave and P-wave velocity profiles developed from all data for Location 1. SPT blowcounts shown for comparison

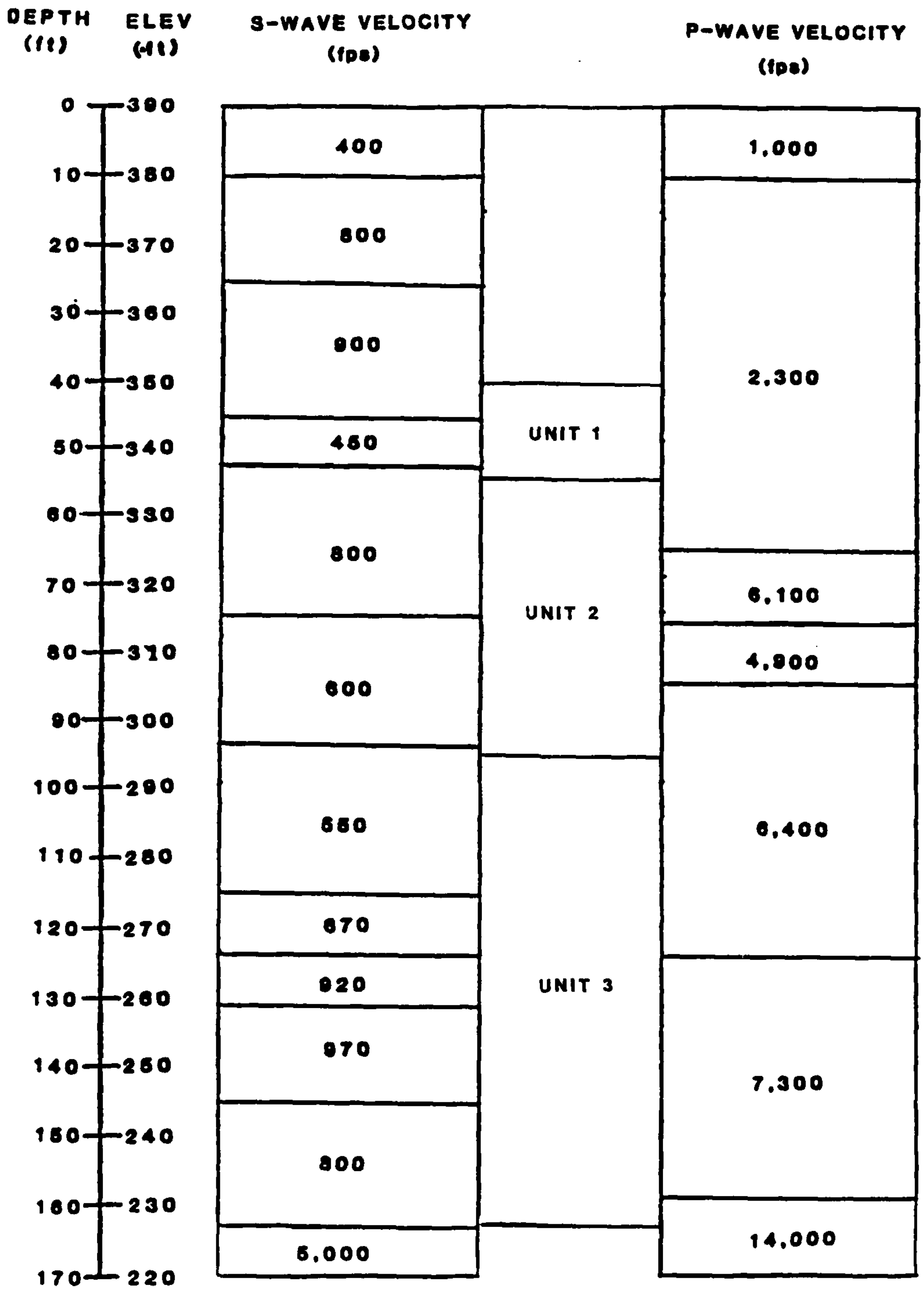


Figure 26. Estimated S-wave and P-wave velocity profiles for dam centerline based on all data at Location 1

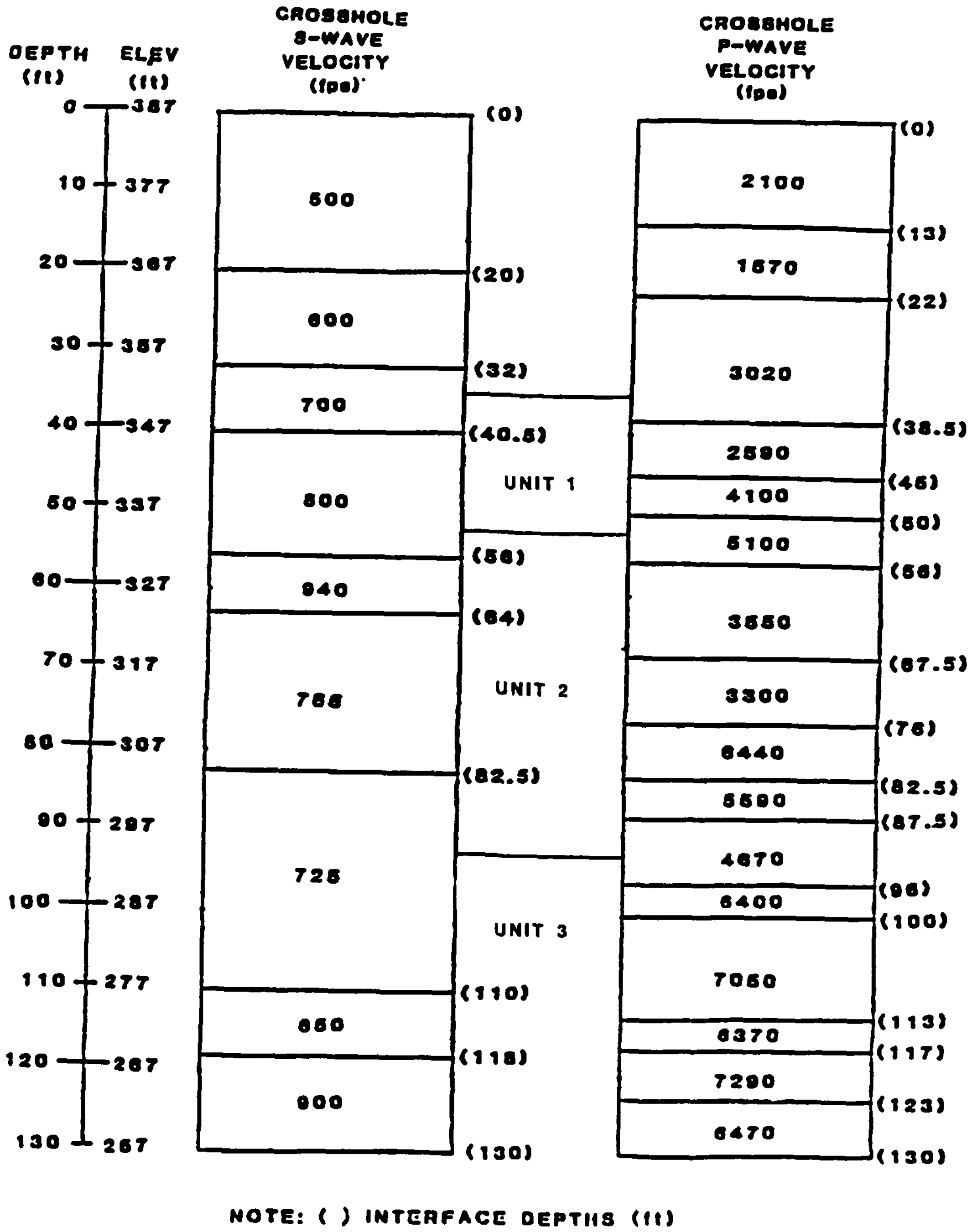


Figure 27. S-wave and P-wave velocity profiles developed from Crosshole tests at Location 2

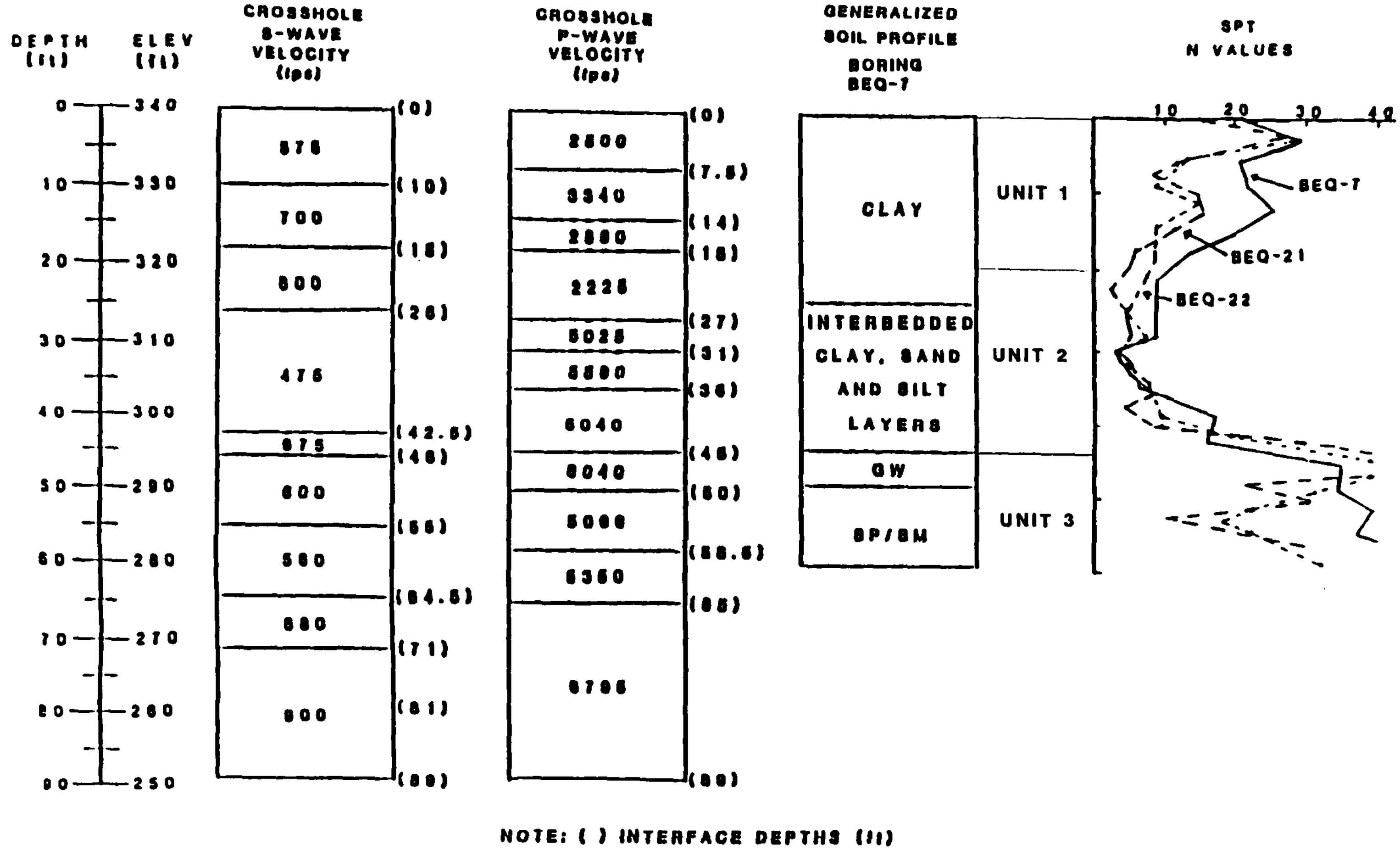


Figure 28. S-wave and P-wave velocity profiles developed from Crosshole tests at Location 3. SPT blowcounts shown for comparison

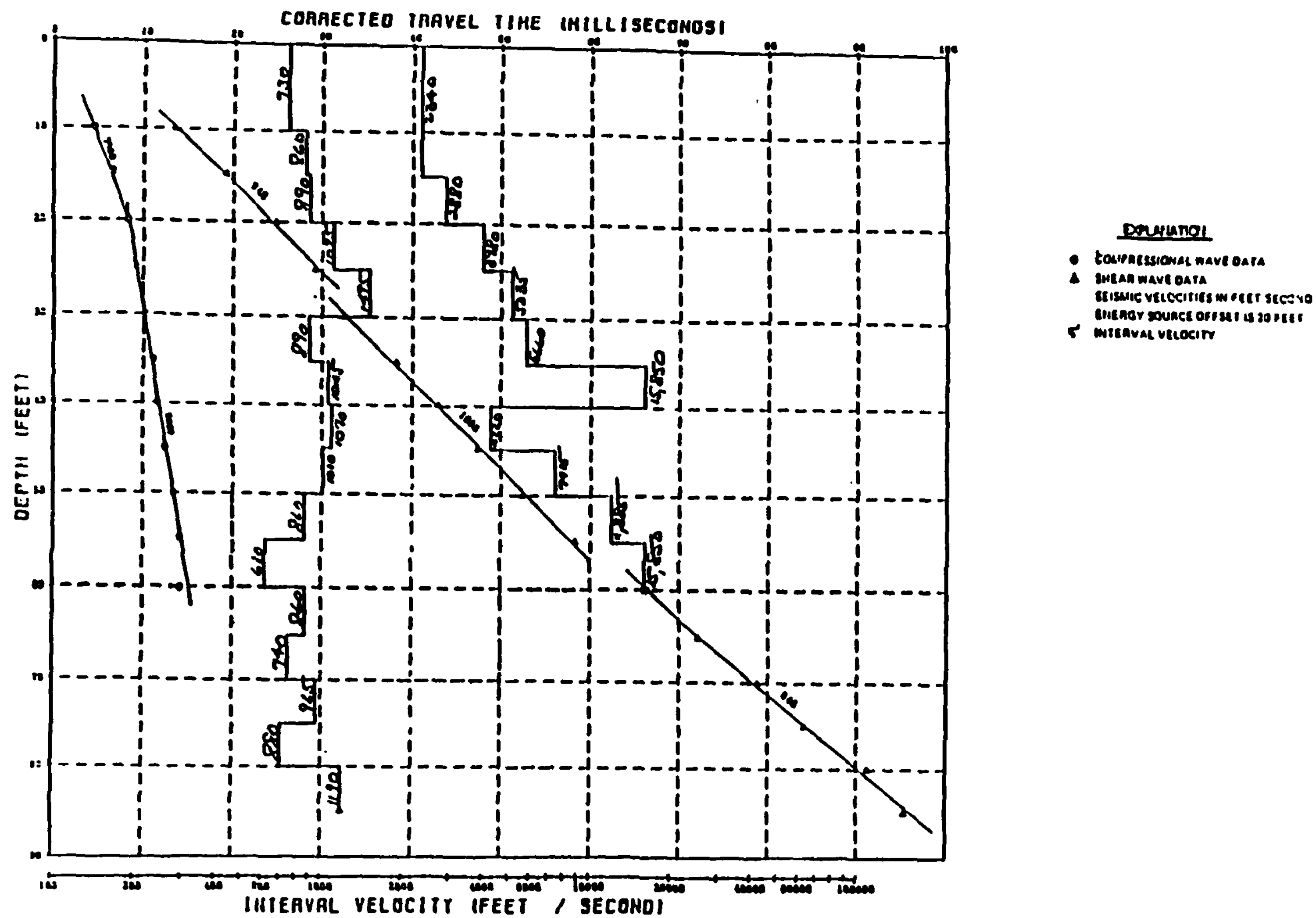


Figure 29. Downhole seismic velocity profile CPT-12 sounding, Barkley Dam, Kentucky

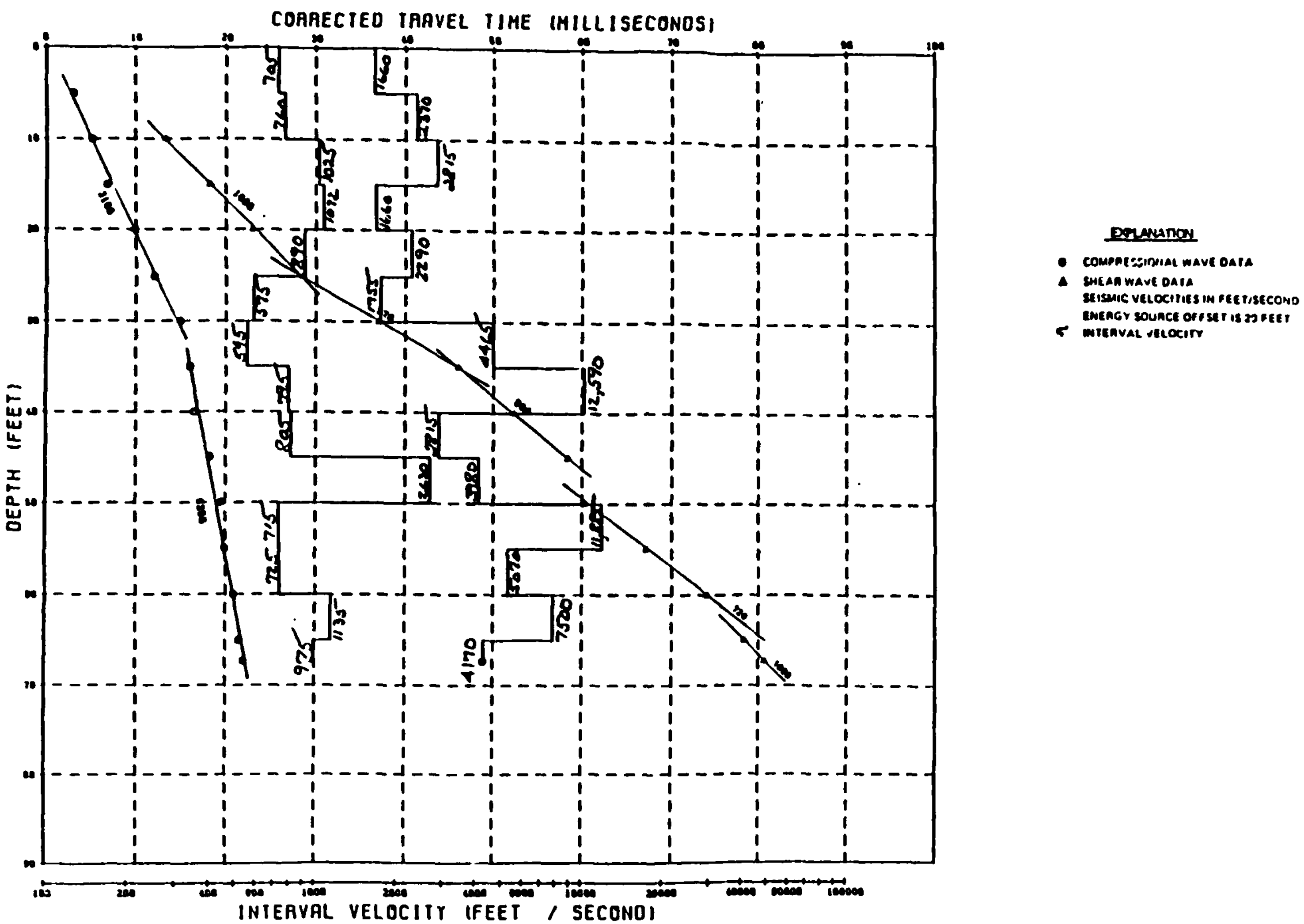


Figure 30. Downhole seismic velocity profile CPT-26 sounding, Barkley Dam, Kentucky

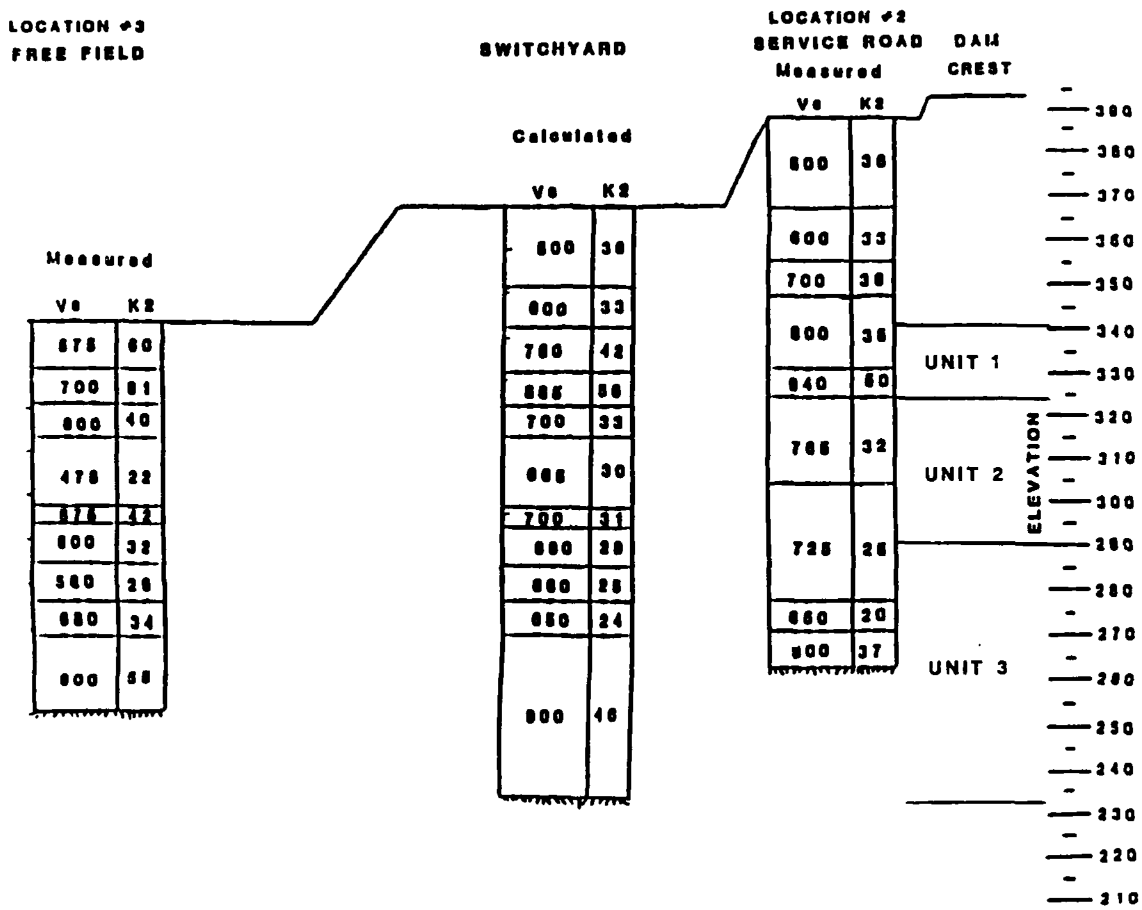


Figure 31. S-wave velocity profile estimated for witchard area, based on averaged K_2 values from Locations 2 and 3

SHEAR WAVE VELOCITIES (ft/s)

DEPTH (ft)	ELV 350	LOCATION 1	LOCATION 2	LOCATION 3	LOCATION 4	TOTAL
0–15	UNIT 1	420-770 665	700-800 765	550-715 620	890-1095 995	420-1095 760
15–55	UNIT 2	485-770 620	725-940 785	445-715 585	610-1495 980	445-1495 745
55	UNIT 3	545-1025 870	650-900 785	560-680 630	740-1190 940	545-1190 806
	MAXIMUM TEST DEPTH	115	127	65	86	
	TEST METHOD	CROSSHOLE	CROSSHOLE	CROSSHOLE	DOWNHOLE	

RANGE AND MEAN SHEAR WAVE VELOCITIES CORRELATED TO UNIT 1-3

Figure 32. Comparison of shear wave velocities, Units 1-3

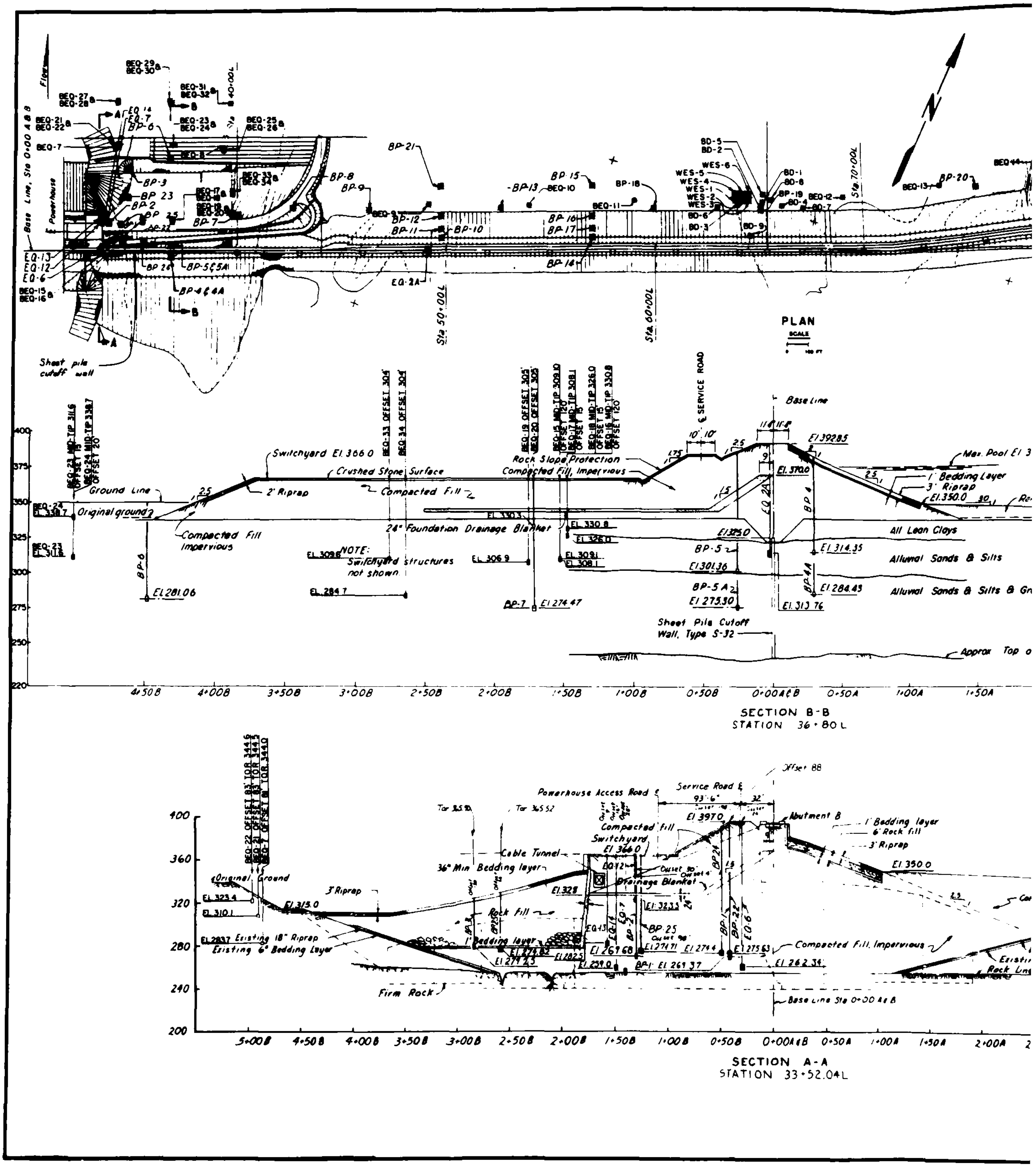


Figure 33. Piezometer locations and sectio

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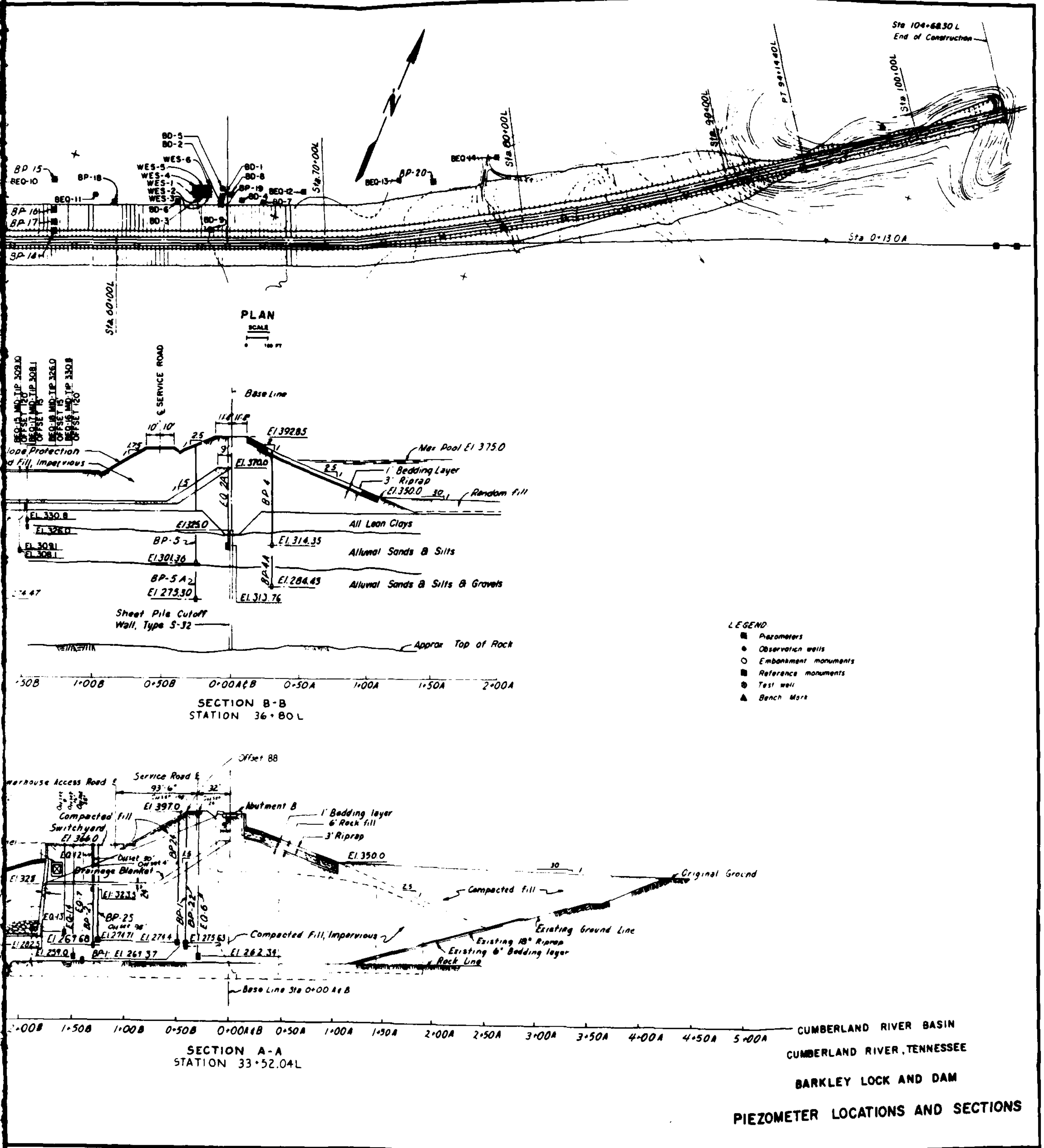

Figure 33. Piezometer locations and sections

2 of

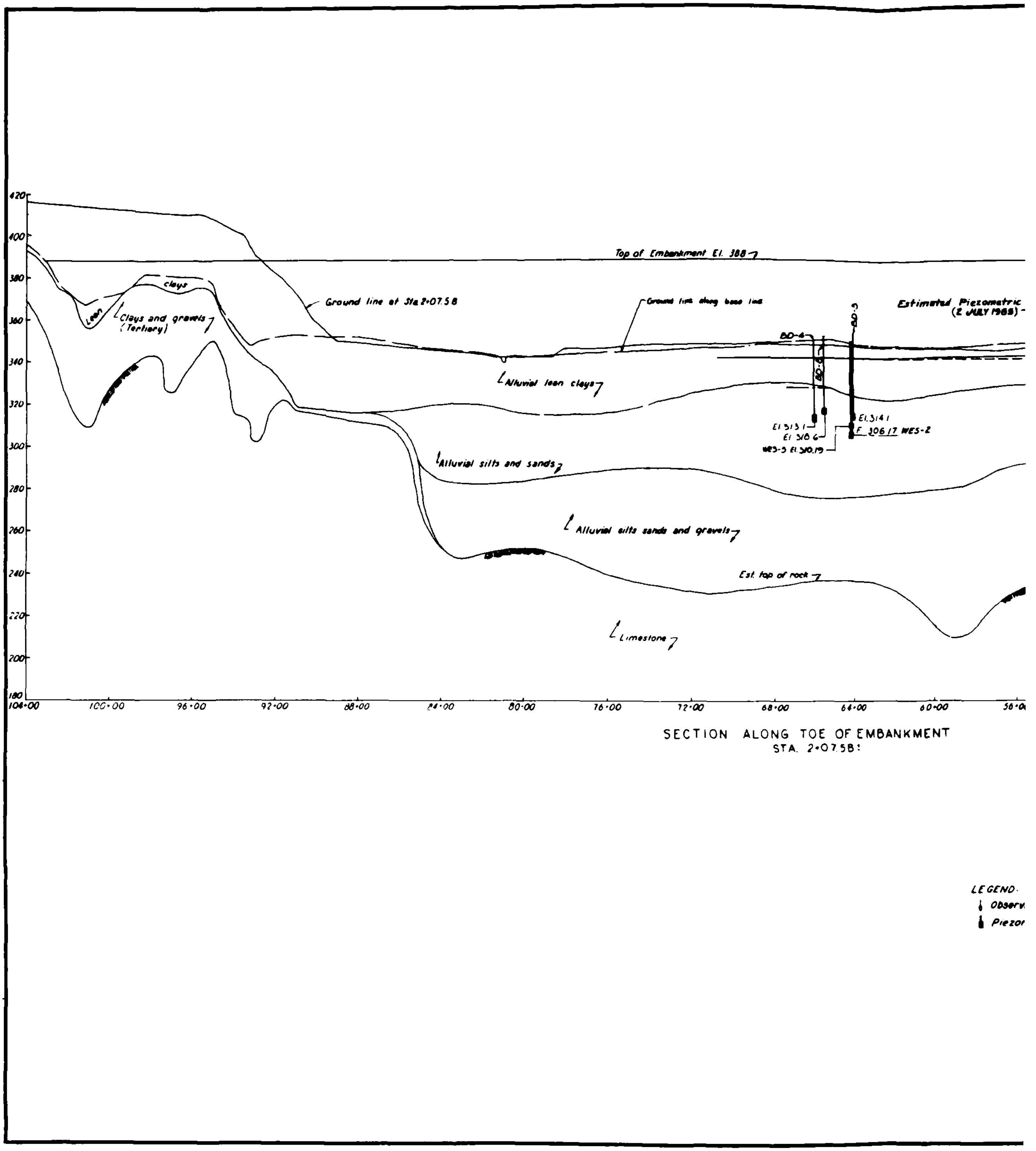

Figure 34. Embankment profile showing piezometric surfaces in Unit 2

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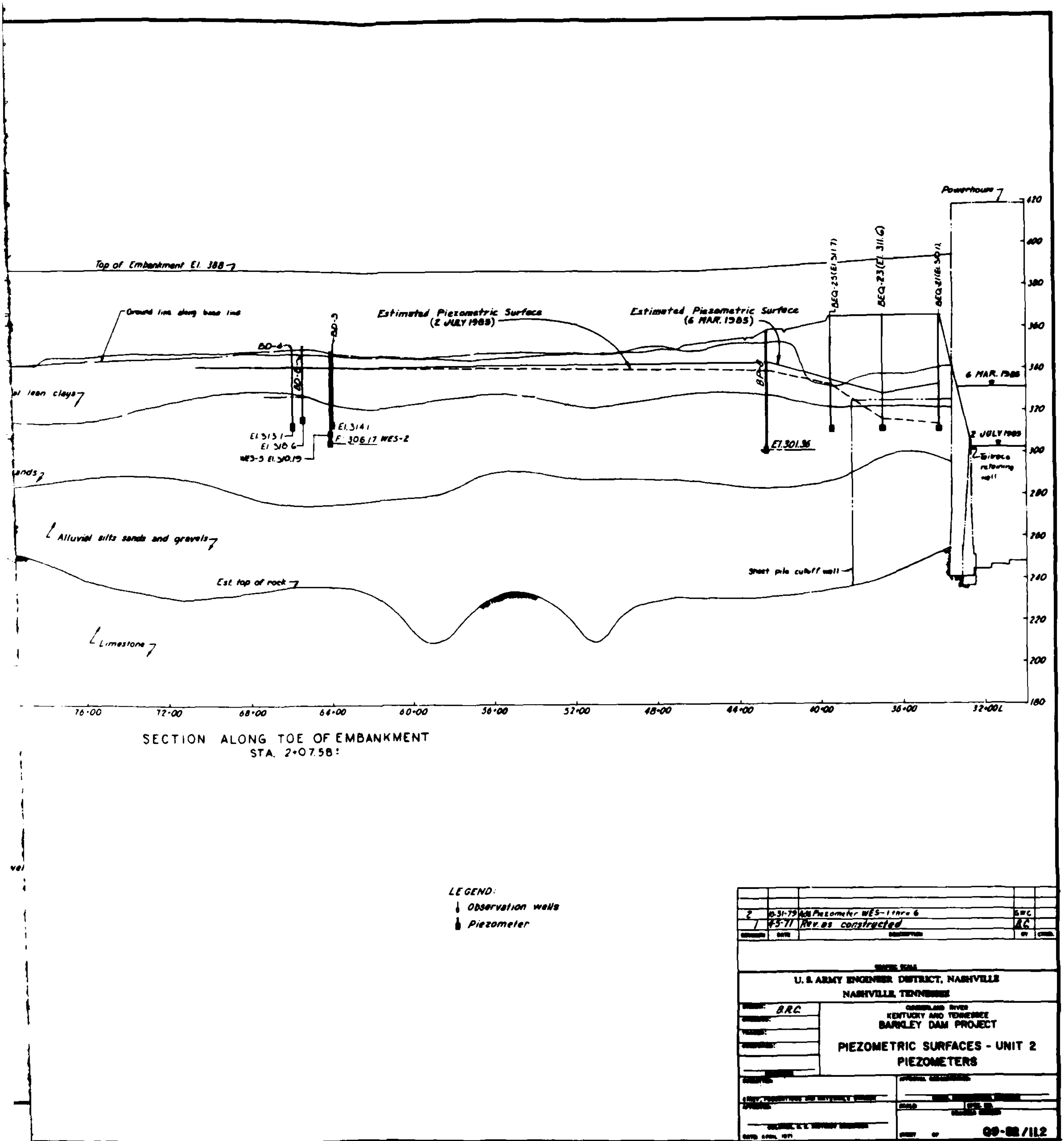


rofile showing piezometric surfaces in Unit 2 for 6 Mar and 2 Jul 1985

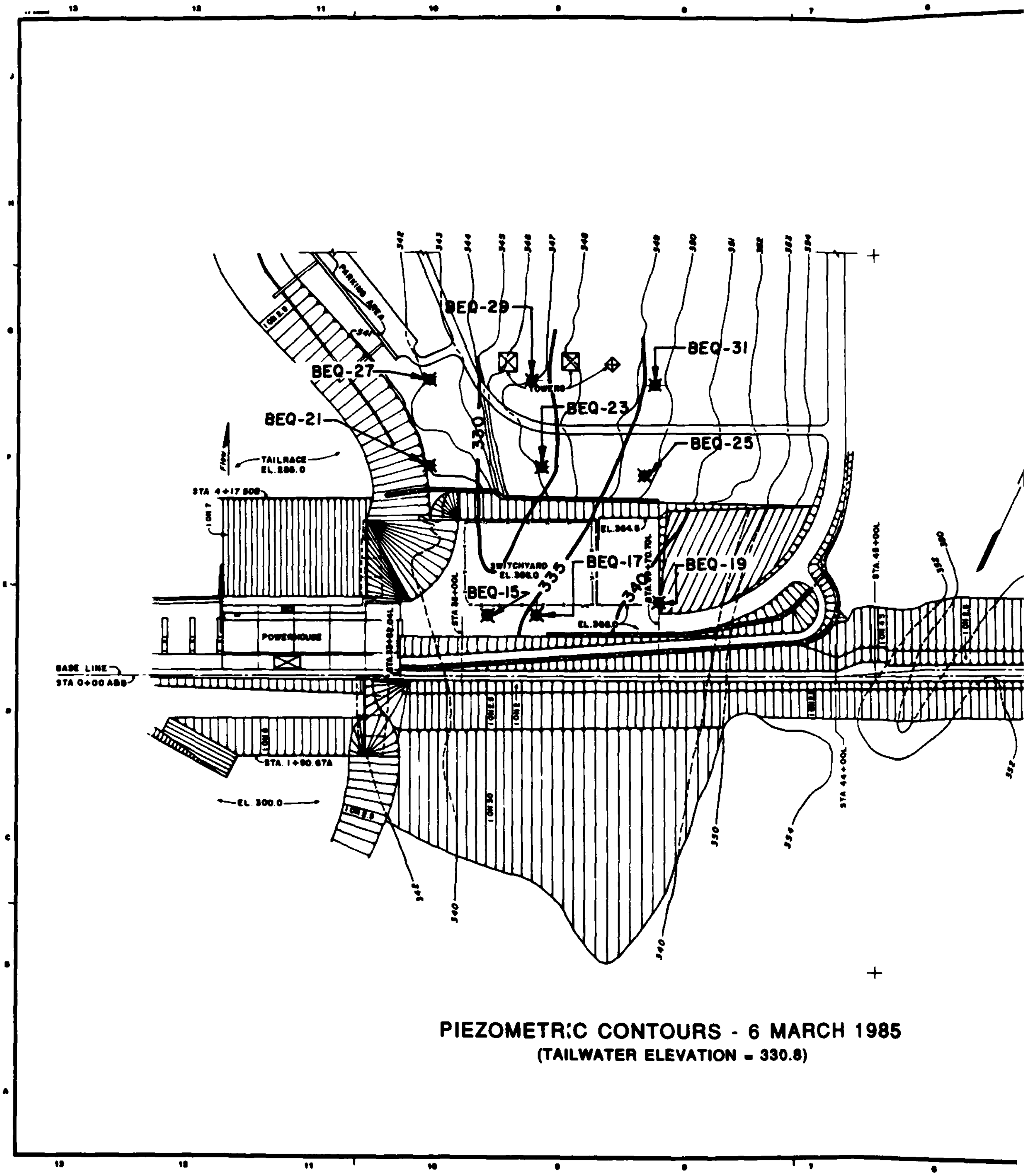
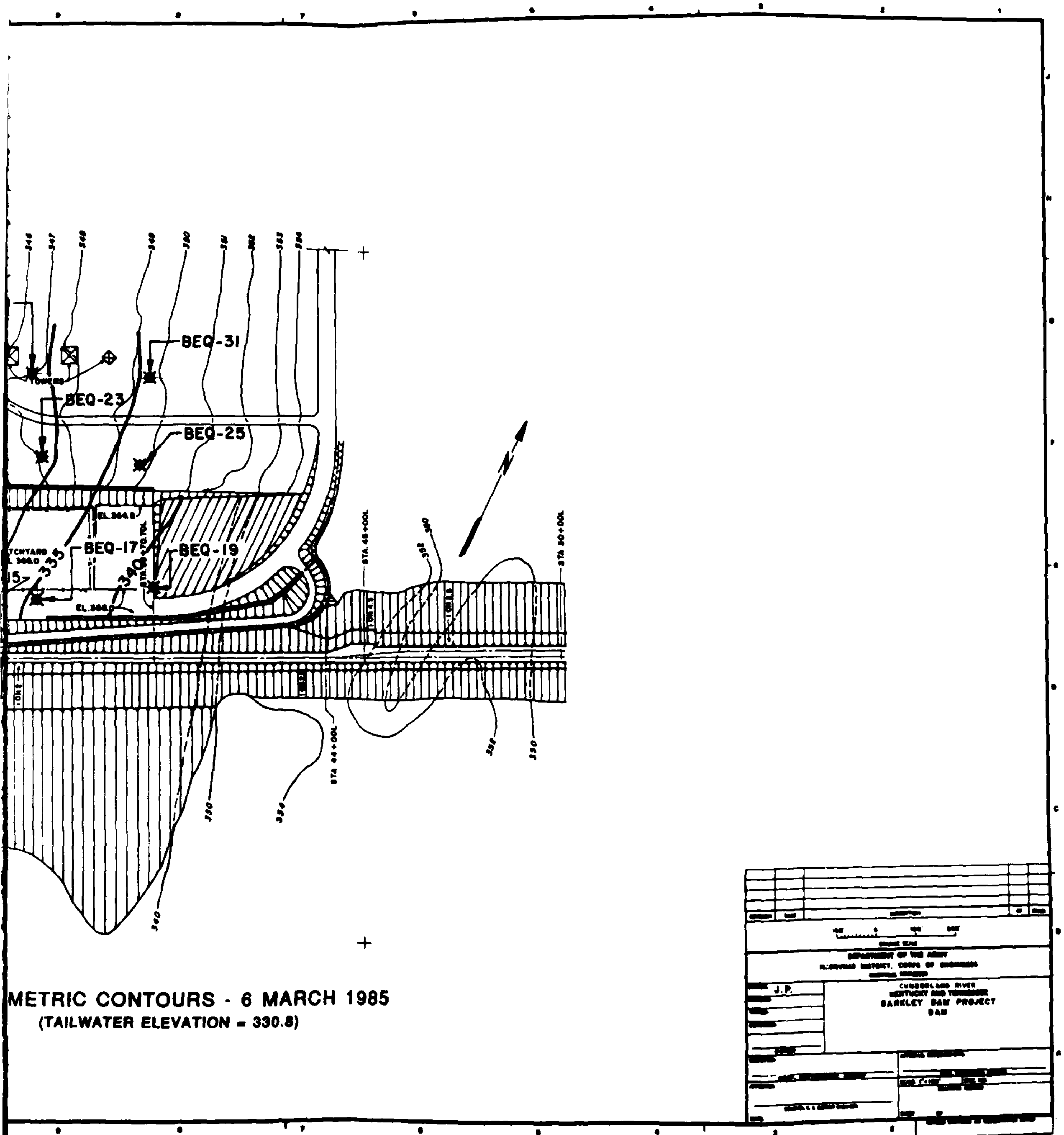


Figure 35. Piezometric contours from readings on 6 M

1 of 2



35. Piezometric contours from readings on 6 March 1985

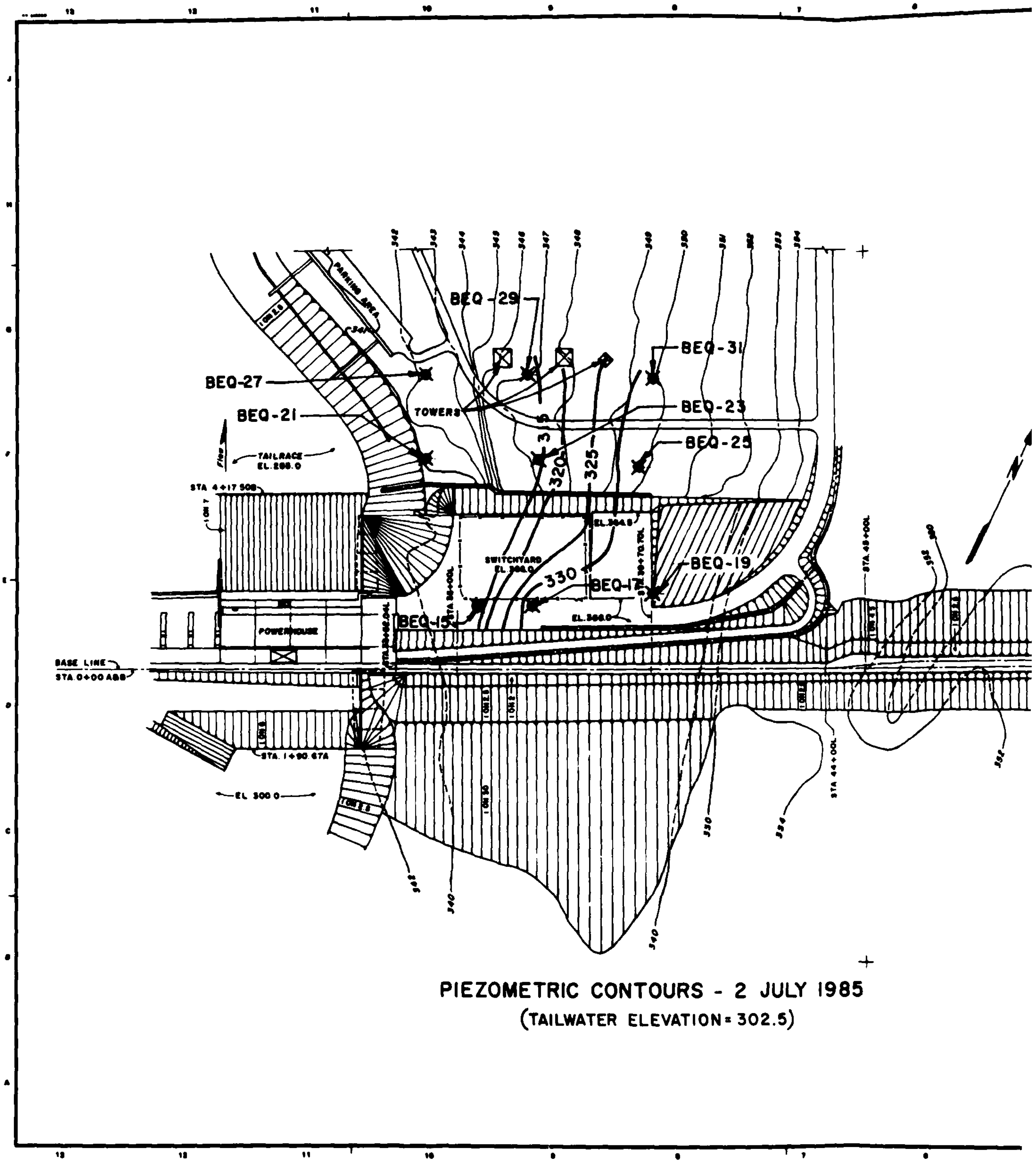
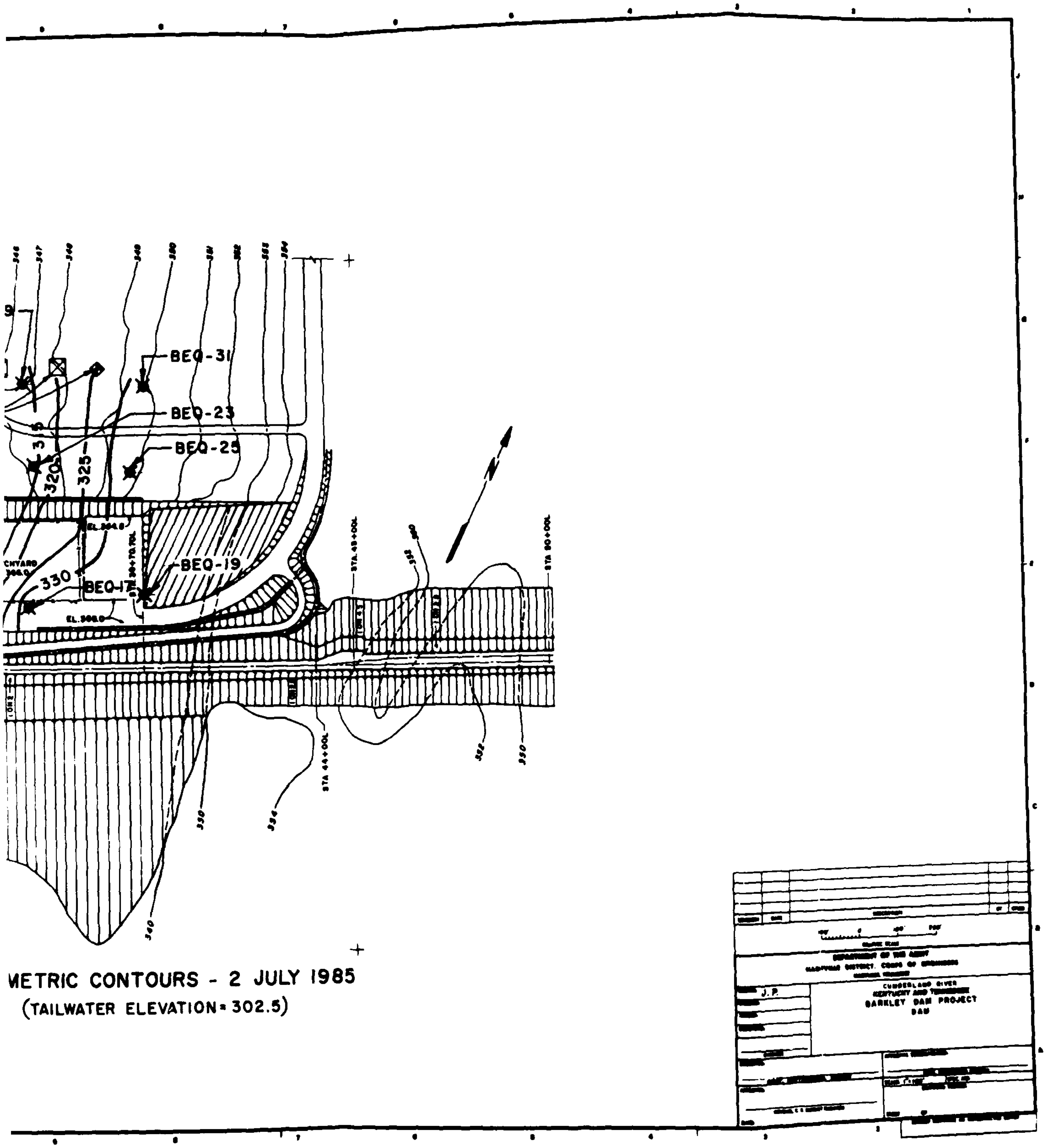


Figure 36. Piezometric contours from readings on 2

1 of 2



e 36. Piezometric contours from readings on 2 July 1985

2 of 2

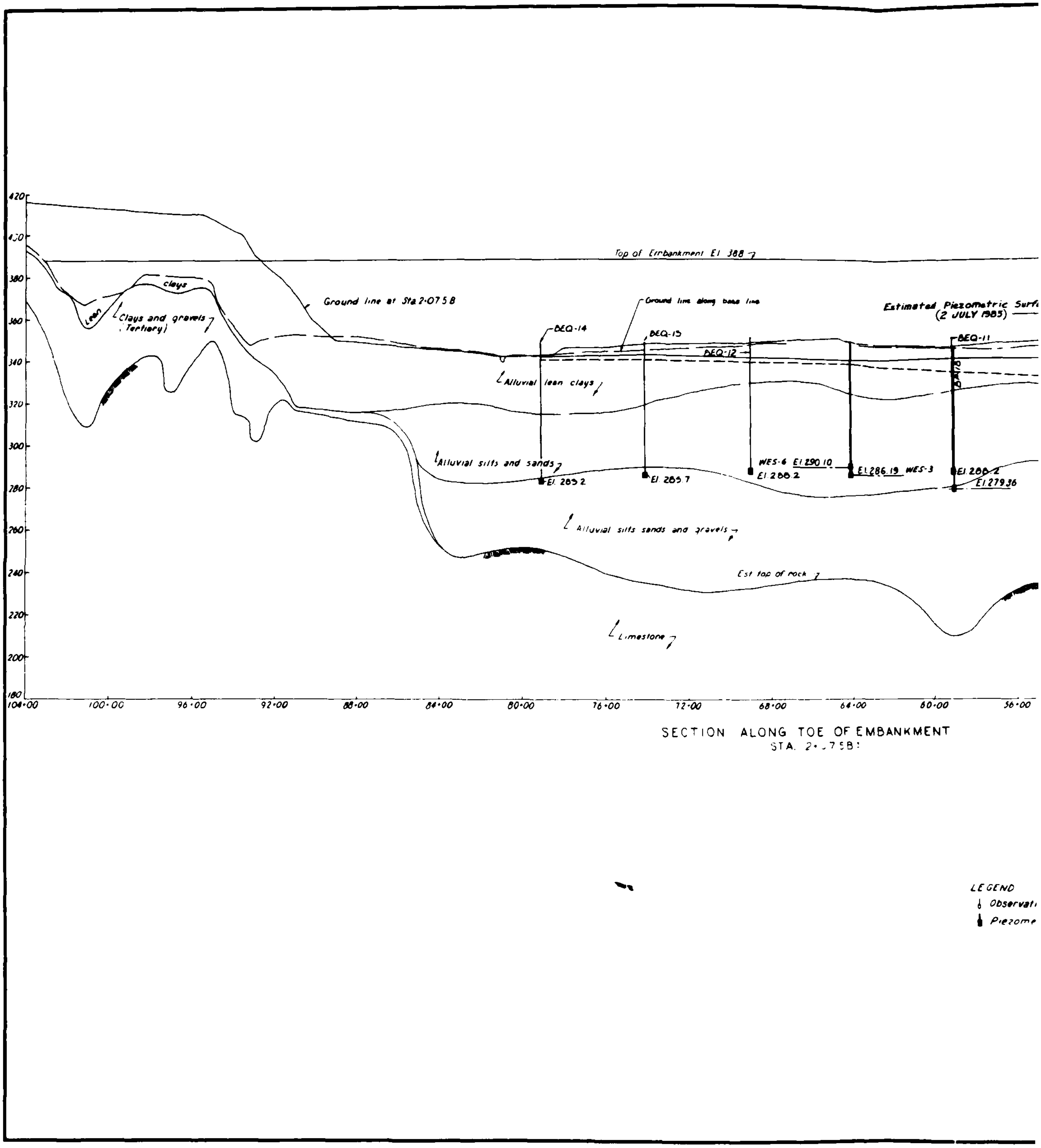
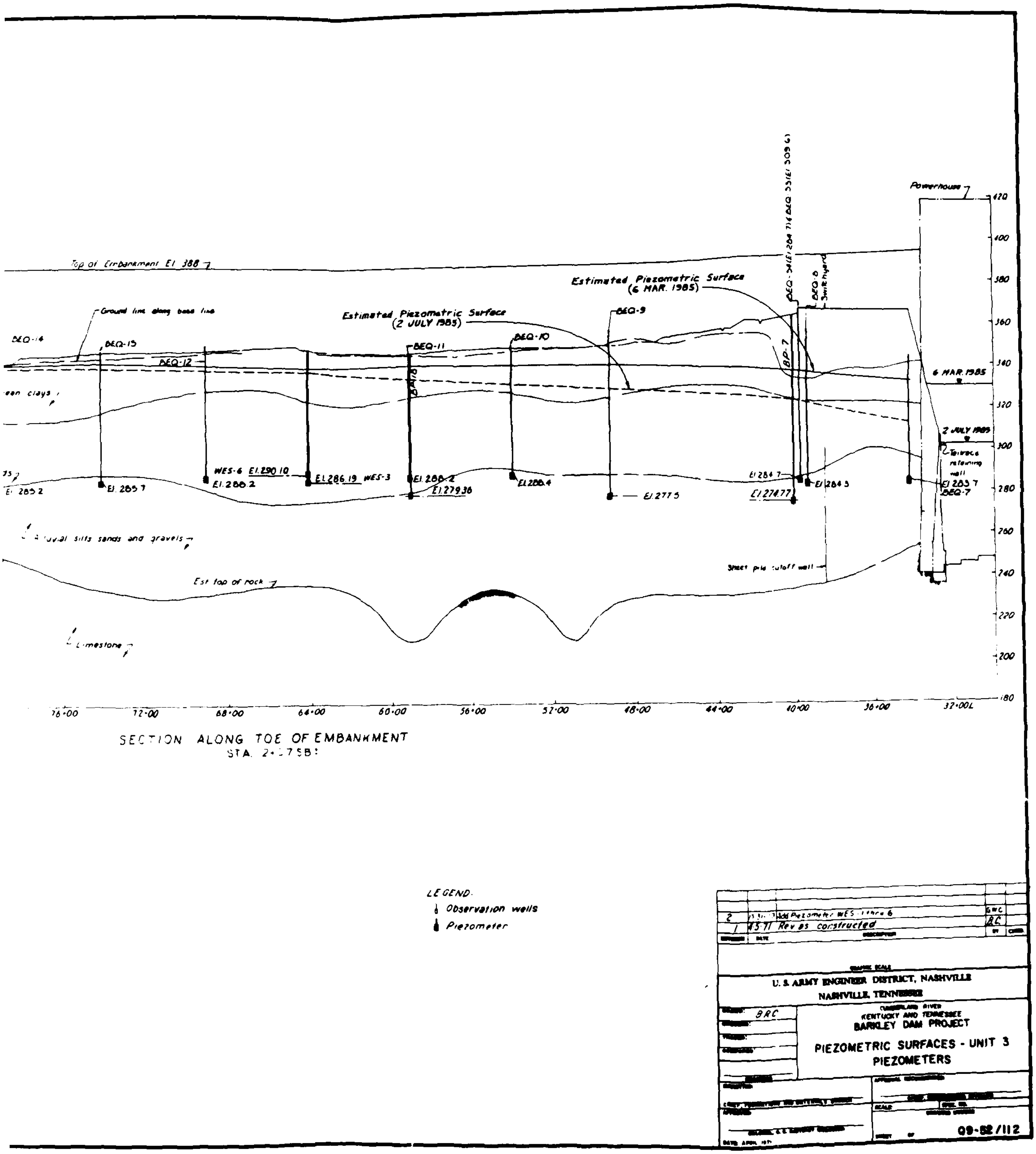


Figure 37. Embankment profile showing piezometric surfaces in Unit 3

1 of 2



profile showing piezometric surfaces in Unit 3 for 6 Mar and 2 Jul 1985

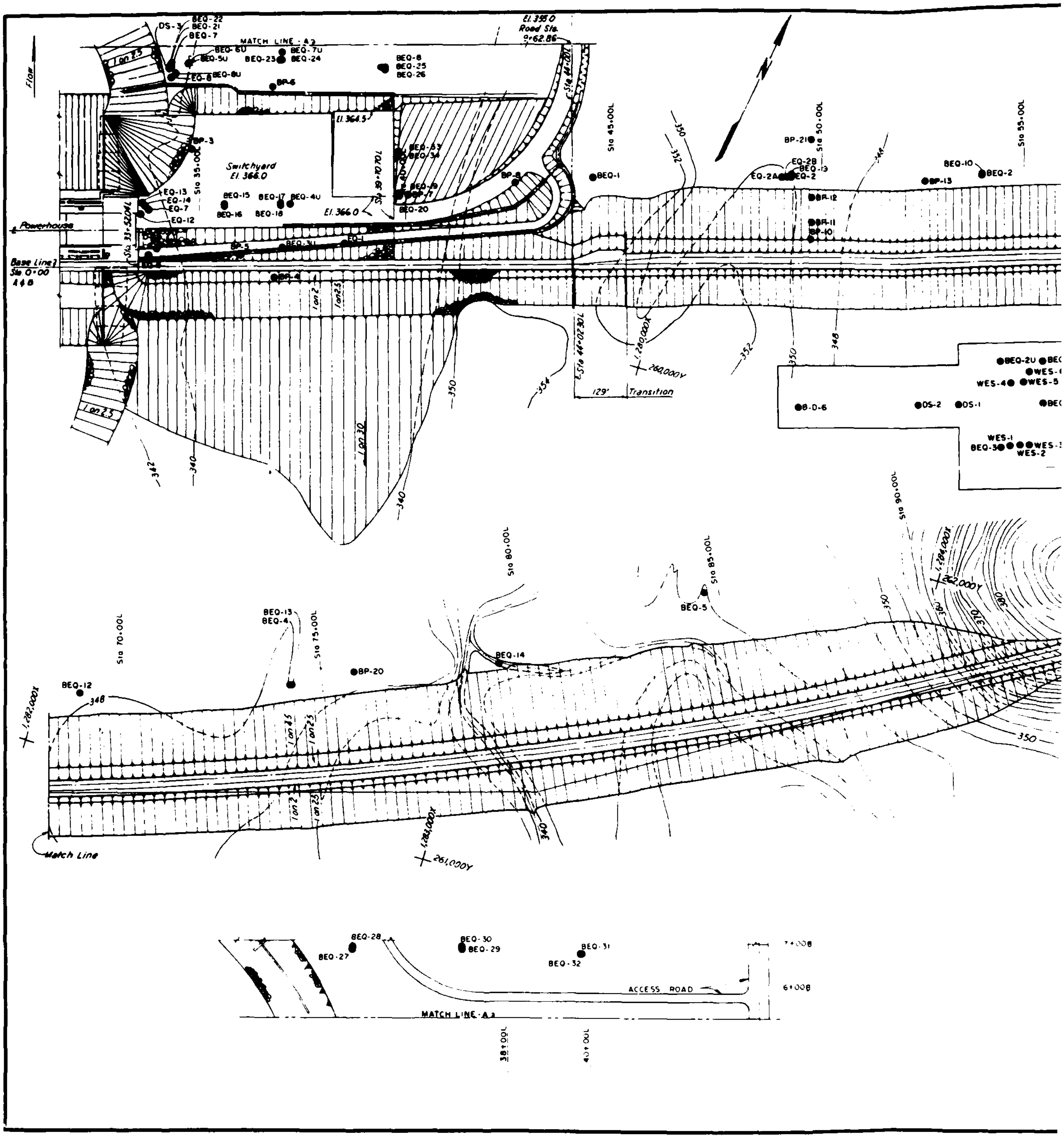
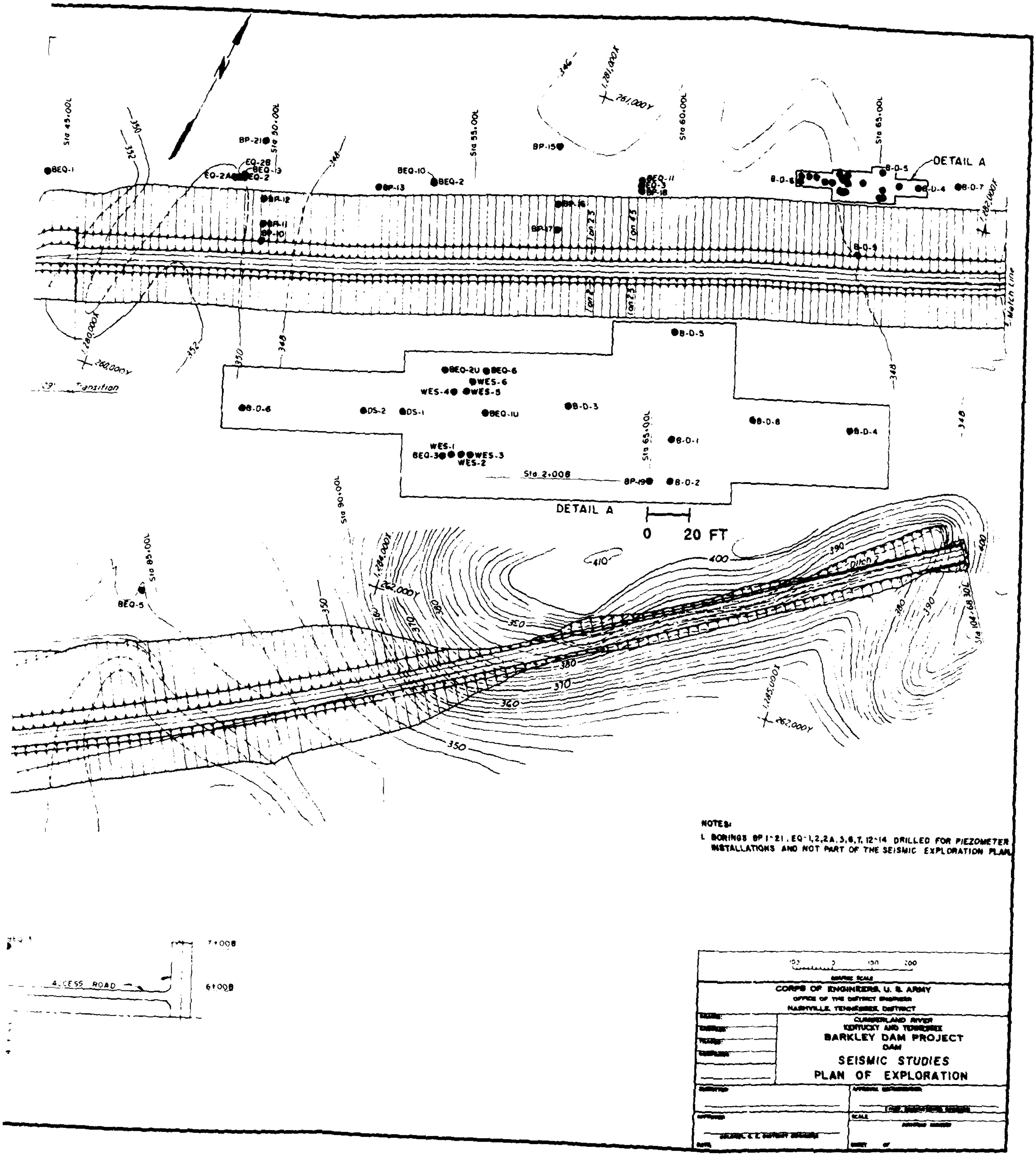


Figure 38. Seismic studies plan of exploration

1 of 2



ɡure 38. Seismic studies plan of exploration

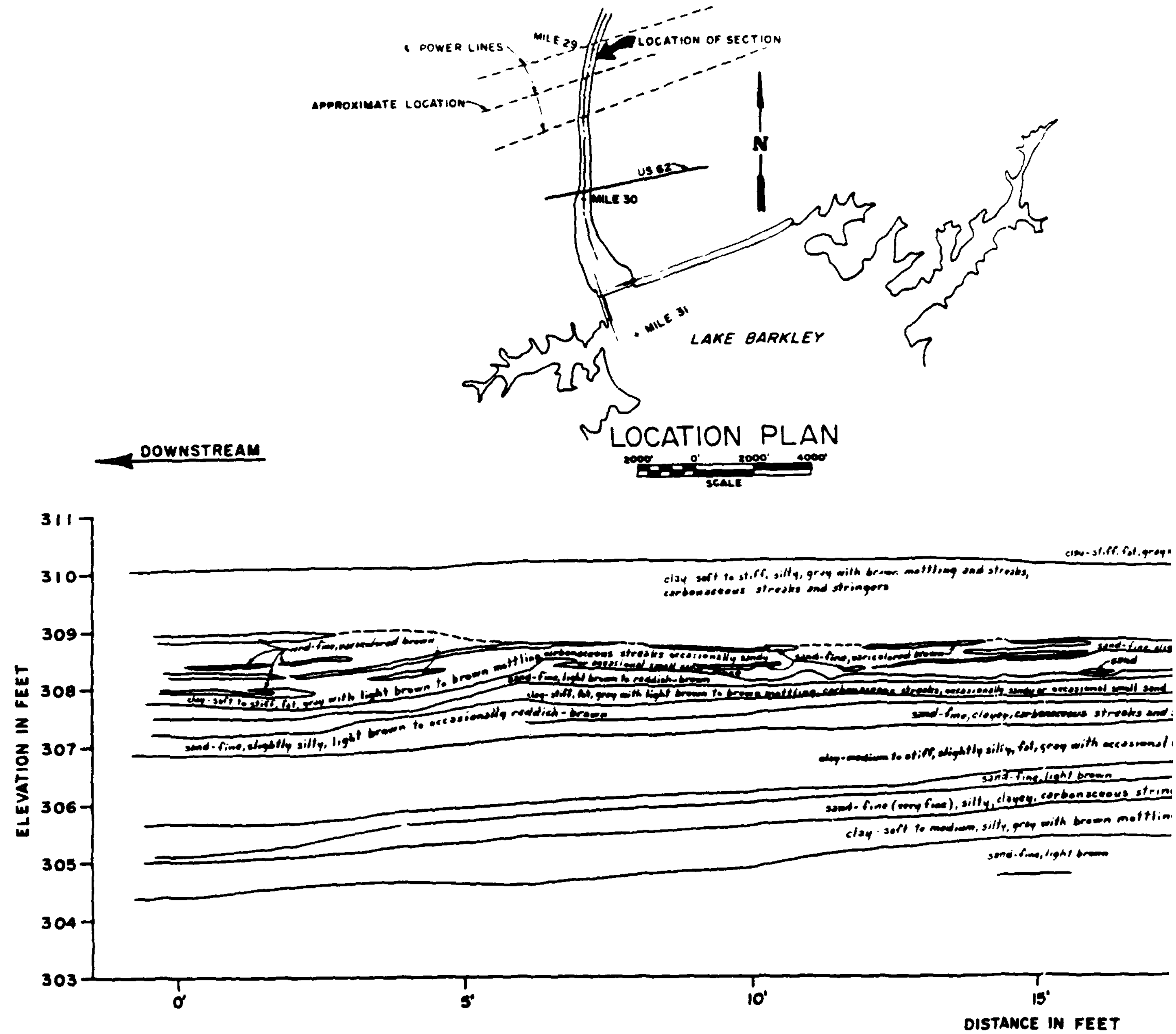


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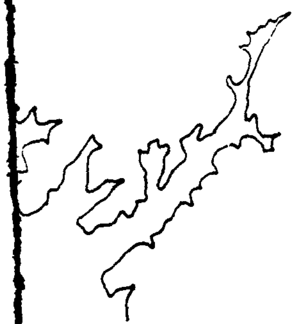
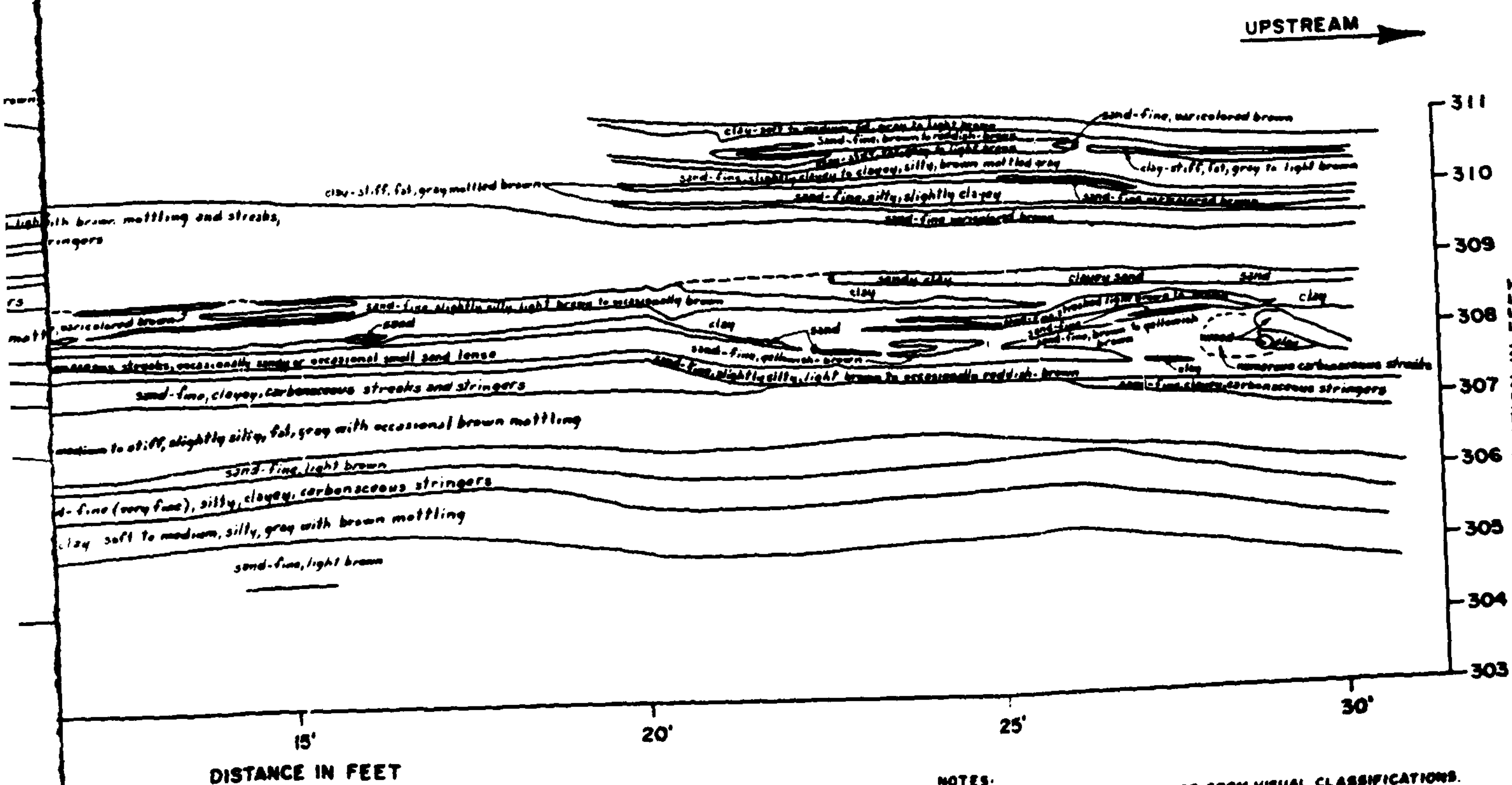
Figure 39. Barkley Dam seismic s





LEGEND

REPRESENTS GRADATIONAL CHANGE

------ APPROXIMATE CONTACT

NOTES:

1. THE ABOVE DESCRIPTIONS ARE FROM VISUAL CLASSIFICATIONS.
2. THE LOCATION OF THE ABOVE SECTION IS ON THE RIGHT BANK ABOUT 1-1/2 MILES DOWNSTREAM OF BARKLEY DAM. SEE LOCATION PLAN.
3. ALTHOUGH NOT SHOWN ON ABOVE SECTION, ONE SAND BED COULD BE TRACED FOR APPROXIMATELY 150 FEET BEFORE IT WAS LOST
4. MAPPING DONE 31 OCT - 1 NOV 1968.

39. Barkley Dam seismic studies

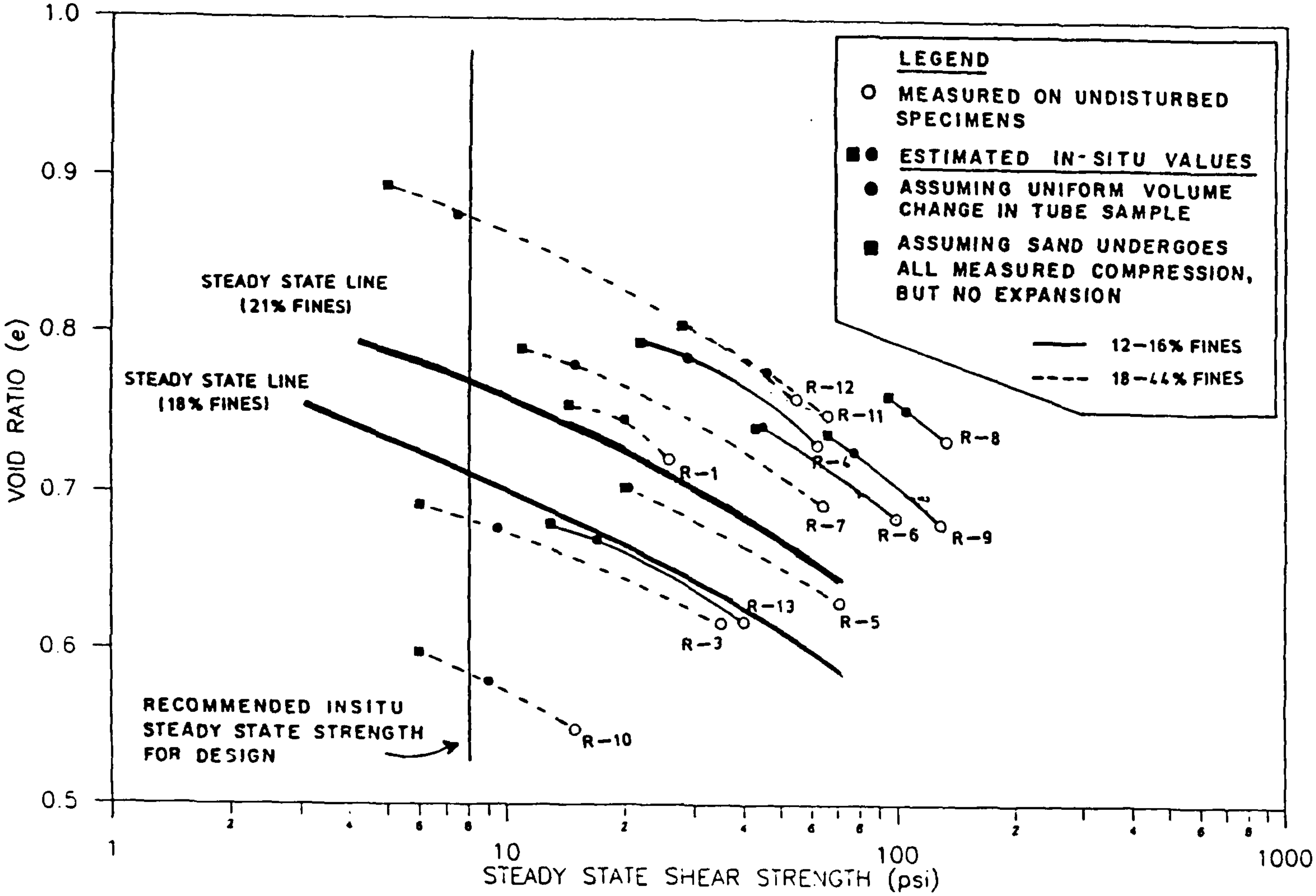


Figure 40. Void ratio versus steady state shear strength from laboratory testing of the undisturbed samples

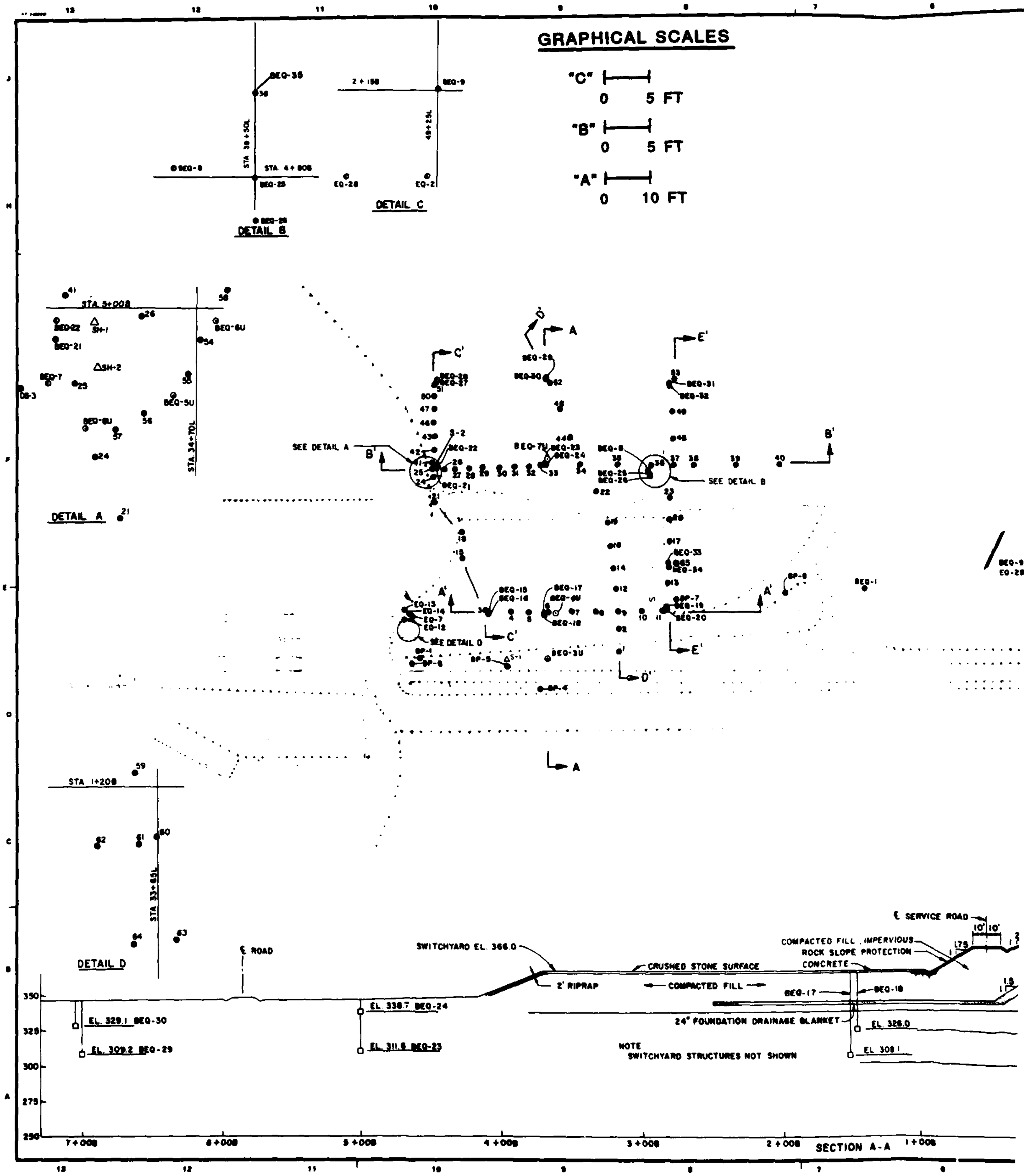
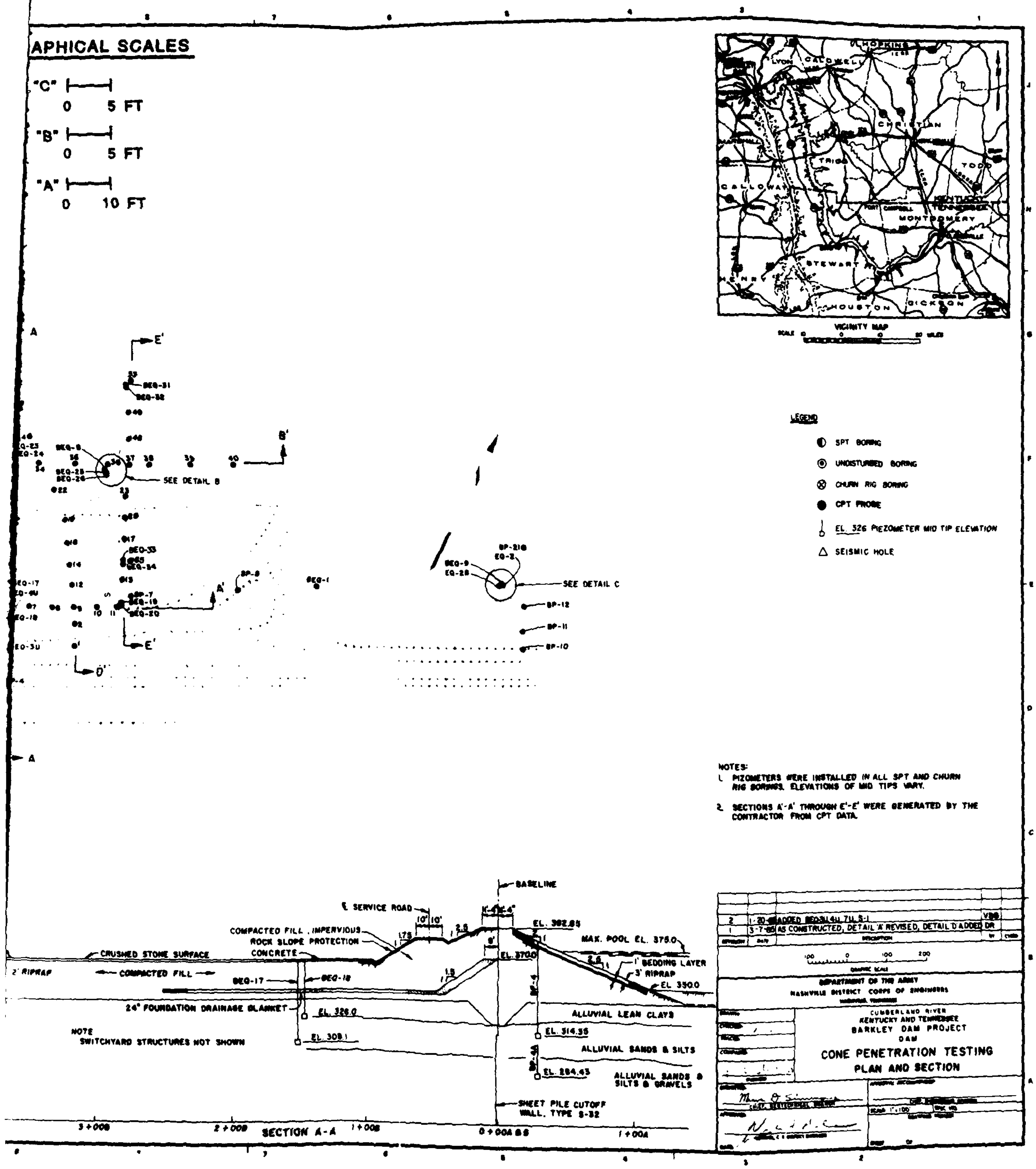


Figure 41. Cone penetration testing plan and



;ure 41. Cone penetration testing plan and section